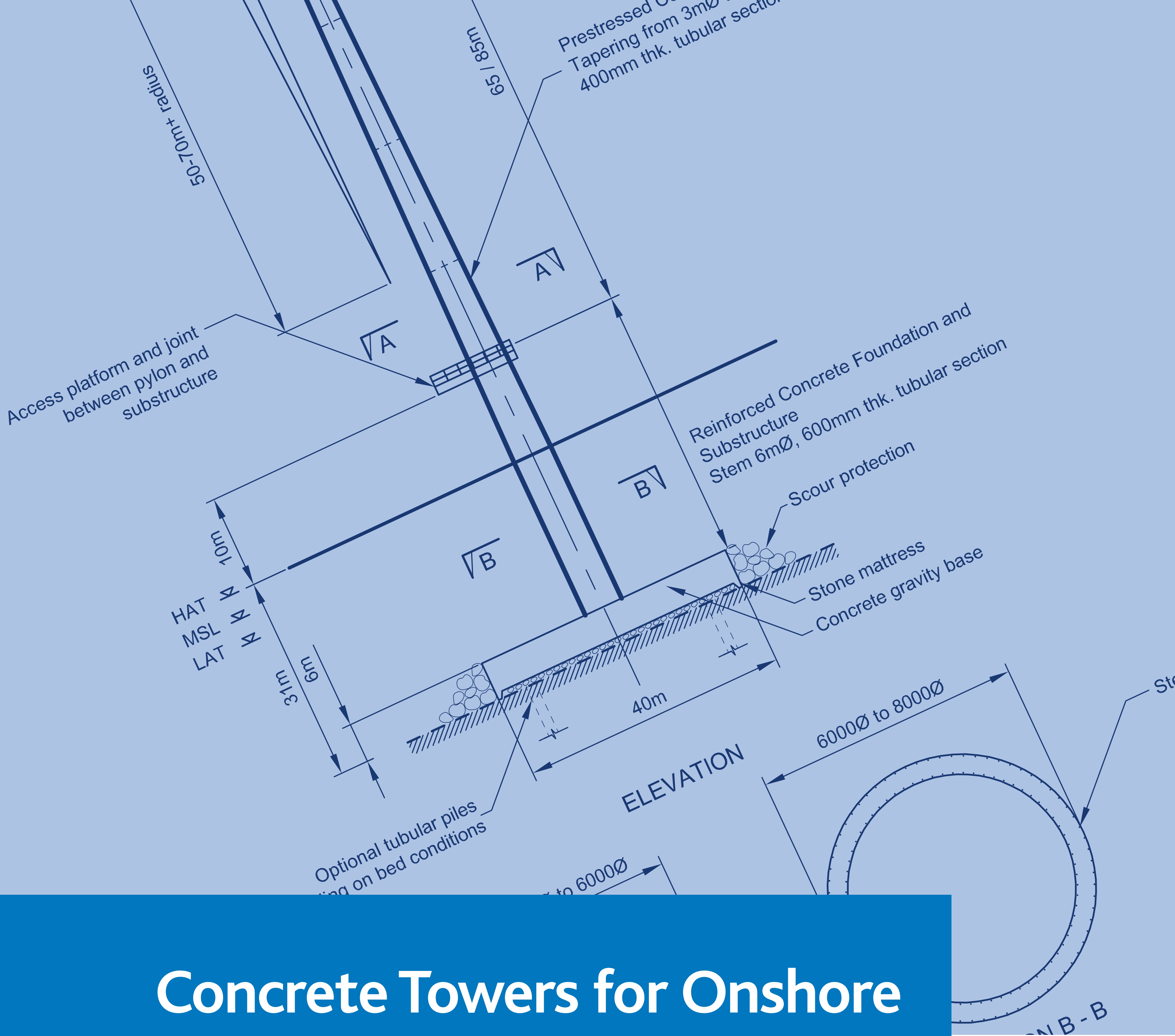

Concrete Towers for Onshore and Offshore Wind Farms

CONCEPTUAL DESIGN STUDIES

Gifford

The Concrete Centre™

PROFILES OF THE CONCRETE CENTRE AND GIFFORD

The Concrete Centre™

The Concrete Centre is dedicated to helping architects, designers, engineers, constructors, developers and clients get the best out of concrete and hence improve their own efficiency.

With a remit to technically develop and promote appropriate concrete solutions, the Centre provides a focal point for innovation, research, development and technical excellence. This is achieved by developing design guidance on a wide range of topics such as structural design, fire, sustainability, acoustics, thermal properties and durability, and providing a comprehensive education and training programme to provide in-depth knowledge and examination of concrete issues and developments.

The use of concrete in the UK wind energy sector to date has been limited predominantly to onshore foundation applications. This contradicts experience from elsewhere, where application of concrete for pylons and offshore applications, such as foundations, is commonplace. As such, The Concrete Centre is committed to challenging the UK wind industry's current thinking on wind tower design and to illustrate how the benefits of concrete construction can be realised more fully.

For more information on the use of concrete in wind tower applications and the services that the Centre offers, including publications, seminars and technical support and advice from our regional teams, please visit **www.concretecentre.com**

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Gifford is a leading engineering design consultancy with over 50 years experience of strategic planning, design development and construction design of major projects for the built and natural environment.

Project experience covers: highways, rail and marine transport infrastructure; industrial and commercial facilities including shipyards, and production plants; energy infrastructure for storage facilities, power stations and wind farms; buildings of all kinds; urban regeneration; environmental management, mitigation and restoration; resources management including waste management.

Founded on the basis of invention of new prestressing systems for concrete and on the innovative design of prestressed concrete structures, the firm has maintained a tradition of innovation in emerging materials, technologies and fields of engineering. These have included pre-fabricated building systems, hovercraft, wave energy systems, wind turbine blades, strengthening techniques for structures and low energy buildings.

Resources include a network of UK and overseas offices with over 600 staff covering disciplines and specialisms relating to civil, structural, building and environmental engineering and science, analysis and modelling, project information production and project management.

PREFACE

Considered opinion supporting concrete as an efficient high performance structural material, in the rapidly growing UK programme for wind energy generation, recently led to the need for verification of this potential. In 2003, The Concrete Centre commissioned Gifford to undertake conceptual design studies of concrete towers for wind energy converters. Studies were subsequently undertaken in two stages during a 15 month period between early 2004 and May 2005.

As offshore sites were perceived to present the greatest opportunities for the UK, initial research focused on this field of study. It is appreciated that, while many common issues exist between onshore and offshore wind tower structures, there are also many important differences that could significantly influence design concepts and feasibility.

This document draws together the results presented in the onshore and offshore research into one publication. It presents ideas and issues on the design and deployment of concrete towers and associated structures, and points to a real opportunity for the substantial and economic use of concrete tower structures in future wind energy developments. We trust that it will help to raise awareness of this important potential.

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1 INTRODUCTION

1.1 Background

Harnessing wind energy is one of the most commercially developed and rapidly growing renewable energy technologies, both in the UK and worldwide. In 1991, the first UK wind farm deployed ten 400kW turbines. Just 14 years later, turbines capable of generating ten times that energy output are in use. Energy generation sophistication has evolved through achievements in engineering design, aerodynamics, advanced materials, control systems and production engineering. Rotor diameters have grown from 30m to over 80m, with diameters of up to 120m now being trialled. Tower heights have risen from 40m to over 100m, and power outputs from 200kW up to 5MW. These dimensions and ratings are set to grow even further.

Looking to the future, the UK is committed to working towards a 60% reduction in CO_2 emissions by 2050. A core part of achieving this aim is the development of renewable energy technologies such as wind. The UK's wind resources are the best and most geographically diverse in Europe, more than enough to meet the target of generating 10% of UK electricity from renewable energy sources by 2010, with an aspirational target of 20% plus by 2020.

The current UK consumption of electricity supplied by the grid is about 35TWh (Terawatt hours) or 35,000GWh (1 Gigawatt hr = 10^6 KW hours). The target of 10% therefore represents delivery of 3.5TWh.

In its recent report *Offshore Wind at the Crossroads* [1], the British Wind Energy Association (BWEA) concludes that, with modest additional government support, offshore wind capacity could rise to 8GW by 2015. By then, Rounds 1 & 2 would be completed and a new Round 3 would have started, taking installed capacity to 10.5GW by 2017. Annual growth rates of installed capacity could be expected to be 1.2GW per year early in the next decade. Other scenarios see this increasing to 2GW per year by 2015 [2].

The table below sets out the effects of this scenario for offshore capacity, combined with current predictions for onshore capacity growth. Capacity utilisation factors for offshore wind of about 36% are assumed, currently growing to 38% over time for new capacity. For onshore wind a utilisation factor remaining at about 30% is assumed.

[The link between rated capacity output is easily calculated, for example as follows:
Current Onshore Rated capacity = 1.0GW Output = 1.0GW x0.30x365x24 = 2628 GWh]

	2006 Capacity GW	2006 Annual Output TWh	% of Total UK Output	2024 Capacity GW	2024 Annual Output TWh	% of Total UK Output
Offshore	0.2	0.7	0.3%	28.3	94.0	26.2%
Onshore	1.0	2.6	0.7%	10.0	26.0	7.2%
Total	1.2	3.3	1.0%	38.3	120.0	33.4%

At the time of writing this report, the rate of installation of offshore capacity is below expectation due to technical, cost and financial issues. It is believed that these issues are likely to be resolved during the next two years, and that actual installation will trend back towards target over the next four to five years. Given the benefits of construction, installation cost and risk reduction, improved turbine reliability and suitable adjustments to the financial support mechanisms, all of which can be reasonably expected, there are good grounds for thinking that UK offshore and onshore wind can contribute some 33% of the required electrical power output by 2024.

1.2 Looking to the future

At present, most recently installed onshore UK wind towers typically have a rotor diameter of 40m, tower height of 70m and power outputs between 1.0 and 2.75MW. The largest individual turbines currently installed offshore (at Arklow Bank) are rated at 3.6MW, with a rotor diameter of 111m and a hub height of 74m. However, even larger machines are likely to be installed over the next few years as various manufacturers release new generation turbines in the output range of 4.5 to 5MW with machines of up to 7MW currently under consideration.

Although these larger generating units are primarily aimed at the offshore industry, experience with some larger onshore prototypes demonstrates the potential viability of onshore exploitation where the constraints of transportation of components can be removed or overcome. To date, a 4.5MW turbine has been installed as a prototype on an onshore wind farm near Magdeburg, Germany. Onshore turbines of this size will require blades in the region of 60m long. They must be supported on taller, stronger towers that stand up to and beyond 100m tall.

Onshore sites generally require taller towers than offshore sites for a given power output. The principal onshore factors include a lower blade tip speed to limit noise generation, leading to relatively larger rotor diameters. The rotor is set at a greater height above ground to overcome greater surface friction effects on the wind speed profile. Onshore towers are currently frequently limited in height by local government planning restrictions to around 100m at the tip of the rotor.

In view of the limited number of available onshore sites with suitable wind climate, location and access, it will become increasingly important to make best use of principal sites by harvesting the optimum proportion of resource available, using the best technologies available. This will put great emphasis on using towers in the range of 100m plus, exceeding existing heights currently used on most UK onshore sites.

In summary, the future trend for UK wind farms is that they are likely to require taller towers, supporting higher powered, longer bladed turbines, many of which may be located at remote or less accessible sites.

1.3 The role of concrete

The consequence of taller wind towers is the need to increase the structural strength and stiffness required to carry both increased turbine weight and bending forces under wind action on the rotors and the tower, and to avoid damaging resonance from excitation by forcing frequencies associated with the rotor and blades passing the tower. In turn this will require larger cross sectional diameters, which may introduce significant transportation problems, bearing in mind that 4.5m is the practical limit for the diameter of complete ring sections that can be transported along the public highway. It is clearly shown in this report that concrete towers can accommodate these requirements and also offer a range of associated benefits.

Concrete is a versatile material and can be used structurally in many different ways. Structural concrete may be ***reinforced*** (with steel bar or other suitable materials), ***prestressed*** (with pre- or post-tensioned steel bars or strands, or other suitable materials) or ***mass*** (with no reinforcement). Concrete used for structures should be regarded as a high performance material. Its properties (particularly, but not only, strength), can be tuned by design over a wide range from normal structural grade to very high performance grades. Mass concrete has a very long history as a construction material. In the modern era, reinforced concrete has been used for at least 100 years and prestressed concrete for over 70 years. Some key benefits of the material are summarised here:

- ***Low maintenance*** – Concrete is an inherently durable material. When designed and constructed properly, concrete is capable of maintaining its desired engineering properties under extreme exposure conditions.
- ***Cost-competitive and economical*** – Concrete solutions can combine low first cost with significantly enhanced life cycle value. Concrete's constituent materials are relatively low cost. The work processes for concrete production are not inherently expensive and are routinely engineered and mechanised to a high level for similar production situations. The potential requirement for the production of significant numbers of similar structures with some commonality of elements also provides a major opportunity for a highly production-oriented design, thereby adding significantly to the potential overall economy.

  For tall wind towers, in particular those standing in excess of around 90m, concrete can deliver cost-effective, long-life solutions. Solutions with a practical design life of 40 to 60 years plus are feasible. This opens up the possibility of significant life cycle cost savings on towers and foundations if coupled with a turbine re-fit philosophy. It may be anticipated that improved technology turbines would be installed at each re-fit.

  Large diameter structures can easily be constructed in concrete without disproportionate increases in cost. In addition to potentially lower relative grid-connection and operational costs, taller more durable towers can generate increased levels of power and deliver lower payback times.
- ***Design and construction flexibility*** – Concrete's versatility enables design solutions with no restriction on height or size to meet challenges influenced by site conditions and accessibility. Designs can be adapted to both in-situ and precast construction methods and offer a wide range of construction flexibility to suit site conditions, availability of specialised plant and labour and other local or market circumstances.
- ***Mix design flexibility*** – Concrete is an adaptable construction material that can be finely tuned through alterations in mixture design to optimise key parameters such as strength, stiffness and density.
- ***Excellent dynamic performance*** – Concrete has good material damping properties. In particular, prestressed concrete has a high fatigue resistance, providing more tolerance and less risk from dynamic failure. By delivering improved levels of damping to vibrations and also noise, concrete designs may play a central role in gaining public acceptance in environmentally sensitive areas.
- ***Low environmental impact*** – Not only is reinforced concrete 100% recyclable, but its embodied CO_2 and energy content can be much lower than that of other construction materials. For instance, for a typical 70m high onshore wind tower configuration, relative to tubular steel, the embodied CO_2 content of a prestressed concrete design option is approximately 64% lower. In addition, a concrete wind tower has the ability to consume CO_2 from the atmosphere both during and after its service life (see Appendix A for further detail).

1.4 Current applications of concrete in the wind energy sector

The use of concrete in the wind energy sector has so far been predominantly in foundation applications, either to form gravity foundations or pile caps. There have been at least two major projects offshore, at Middle Grunden and Rodsand wind farms, which both use gravity foundations with 'ice cream cone' stems to suit the particular conditions of the Baltic Sea. The associated Danish energy production and distribution company ENERGI E2 have declared significant cost savings through the use of concrete gravity foundations for offshore wind farms and reportedly intend to exploit this potential for future sites such as London Array [3].

In terms of pylon applications, concrete solutions are being exploited onshore by at least three turbine manufacturers: Enercon, GE Wind and Nordex.

Enercon [4] has progressed furthest with the development of concrete pylons, with a prototype precast concrete solution now having moved into full scale commercial production. Enercon now offers the option of a concrete tower solution for turbines with hub heights of 75m and above. For hub heights of up to 113m, reinforced precast concrete rings of around 3.8m high with wall thickness of around 350mm are used. The rings range from between 2.3m and 7.5m in diameter, with the larger lower rings split in half vertically to simplify transportation. Once assembled, the concrete rings are post-tensioned vertically. Enercon has recently opened a dedicated plant to produce these precast concrete segments in Magdeburg, Germany. Enercon proposes an in-situ concrete solution for towers standing above 113m high, although no information on this is available at the time of publication.

GE Wind [5] is currently investigating the viability of hybrid concrete and steel pylons. A 100m tall prototype constructed in Barrax, Spain, features in-situ, post-tensioned concrete rings for the bottom 70m and a steel tube for the top section. The concrete section, which uses diameters up to 12m, is post-tensioned vertically with the cables running inside the tower, but external to the concrete wall. Again, a conventionally reinforced section with wall thicknesses of 350mm is used. The steel section is split into five sections, each weighing a maximum of 70 tonnes, with a maximum diameter of 5.7m.

A joint venture between the two Dutch companies Mecal and Hurks Beton [6] has produced a design for a 100-120m concrete-steel hybrid tower which is the subject matter of several written papers. The solution is similar to the GE Wind hybrid tower, in that it uses post-tensioned concrete, with the tendons inside the tower but external to the concrete section for the lower concrete section and a steel tube for the top section. However, the design makes use of long and narrow precast concrete elements rather than in-situ rings. Again, these elements are conventionally reinforced, with wall thicknesses of 250-350mm.

Clearly, Enercon believes that the precast ring segment solution is economically viable for heavier turbines for large towers. Indeed, the company has invested heavily in a production plant and is producing a stock of segments. Mecal and Hurks Beton [7] also believe that a concrete/steel hybrid tower is economically viable, focusing on whole-life costing for a wind farm and suggesting that although the initial investment in the hybrid tower will be greater than a steel tower, the returns will be greater because the tower is taller and generates more power.

As of August 2006, Nordex is offering concrete/steel hybrid towers for hub heights of 120m. Previously, it used solely steel towers but has recognised that concrete offers a relatively inexpensive alternative [8]. The solution comprises a 60m length of modular steel in three sections on top of a concrete tower produced in different lengths to provide hub heights between 100m and 120m. This involves the use of locally supplied materials and ensures an optimum turbine height to make the most of prevailing conditions. Nordex also recognize that this approach offers logistic advantages as the restrictive steel diameters normally applicable during transportation do not apply.

1.5 Background design considerations

Design concepts have been developed involving the use of concrete for wind towers, with the aim of achieving cost competitive and practical solutions for UK conditions. During the evolution of these concepts, information has been published on a number of other European designs that are currently available or under development. In the structural concepts set out here there are, unsurprisingly, some features shared with these other designs. The process has been to develop a synthesis of ideas and concepts arising from a fundamental consideration of the requirements and opportunities by the study team. It is hoped that this will point to some new directions and, at the very least, confirm and highlight important existing solutions which are worthy of greater consideration and application in UK practice.

Design solutions for offshore and onshore wind pylons have many similarities. Whilst some of the design and production thinking and subsequent development experience could be interchangeable, there are nevertheless some profound differences (see Section 7). One obvious difference is that offshore structures are subjected to additional loadings from waves and currents and to generally more aggressive conditions. Furthermore the construction and operating regime offshore is considerably more severe and hazardous. Specialist heavy plant needed for work offshore is subject to serious and, to some extent unpredictable, disruption from bad weather.

Early wind farms and turbine technologies were developed in the more benign conditions prevailing onshore. To date, concrete has been used much more widely in onshore wind energy structures than in offshore wind farms. Foundations for onshore wind towers are already predominately constructed using reinforced concrete, either as gravity bases or for

caps over piles. Only a limited number of wind pylons worldwide have so far been constructed using concrete, and these have been onshore. In the UK, examples of any size are restricted to one or two large towers for early prototypes.

The overwhelming majority of wind towers constructed to date have been built using tapered steel tubes formed from seam welded rolled plates with flanged bolted connections at the terminations. Taller towers were built up from separate lengths determined by transport and lifting constraints. For the larger towers specialist transporters are required to carry the tower sections from the fabrication yard and maximize the ruling diameter of the tube within the highways loading gauge. This has generally limited maximum steel tower diameters to 4.5m. However, it is understood that work is underway to develop segmented designs to overcome this limitation. This will require the introduction of costly bolted joints into the thickest and most heavily loaded sections of the tower.

Tall concrete towers and chimneys have a long and successful history and, whilst there are good reasons why steel towers have been the dominant solution to date, issues associated with increasing height, diameter and loading tend to point towards the use of concrete tower designs providing alternative, and potentially more competitive, solutions for future wind farms.

It would seem that the onshore industry is moving to a position where there could be substantial progress on the widespread use of concrete towers. Since insitu concreting is currently utilised for all onshore sites, logistically it would not be a big step to use slipforming. This is a particular in-situ technique which is entirely crane-independent. It can be used in conjunction with precast concrete or steel elements prefabricated off-site, to provide an efficient way of overcoming transportation limitations that would otherwise arise with the very largest elements such as structural towers. Slipforming can be used to construct tapering towers of any height desired.

The outcome is that the introduction of concrete-related construction techniques into the pylon element of towers, either in lower pylon sections or over their entire height, is a relatively simple and logical design step.

1.6 Towards more competitive concrete design solutions

Concrete wind tower solutions must be cost competitive with alternative and existing design options. For typical wind tower heights of 60-80m constructed to date, it is difficult to achieve designs and construction approaches where the lower specific cost of concrete as a material offsets the required increase in material quantity and weight. Even by adopting radical design approaches to minimise the thickness, and therefore the weight, of concrete towers, this approach is still likely to be significantly heavier overall than a steel design. As construction of next-generation wind farms with towers up to and beyond 100m high comes into closer focus, and given an expanding programme of construction, a number of factors make concrete an attractive design option for delivering large diameter pylons at acceptable cost.

An increase in tower weight may cause a number of significant effects. Beneficially, the inherent weight and stiffness of concrete towers can offer improved fatigue and dynamic performance levels and may also improve the efficiency of the foundation in resisting overturning forces. By careful attention to the distribution of weight over the height of the tower, the potentially adverse effects of this weight on the dynamic characteristics (natural frequency) of the tower can be minimised. The ability to tune the stiffness relatively easily by adjustment of the lower profile and thickness of the tower shell can more than offset any disadvantages.

Adversely, there may be a need to use larger, more powerful plant both for transportation and for construction, both of which are less readily available and more expensive. Ideally the optimum size of cranage for erection should be determined by the nacelle weight (typically in the range of 70-150Te depending on the turbine rating). These issues may be more easily resolved by using in-situ concrete techniques (including those mentioned above) or precast concrete solutions that lend themselves to being split into a greater number of segments, or combinations of the two. While an increased number of segments might result in an increased number of transport movements and some increase in construction time, this approach may offer a cost-effective solution to site constraint challenges.

Fundamental characteristics of concrete in its freshly produced state are its plasticity (mouldability) and relative tolerance of handling. These make it extremely adaptable to a wide range of construction methods and to the production of complex or curved shapes. A variety of in-situ methods are available in addition to precast solutions. If hybrid concrete solutions (the combined use of precast and in-situ concrete) are considered, the adaptability of solutions increases even more.

The characteristics of concrete as a high performance structural material are of course well demonstrated in the huge inventory of major and complex structures around the world, both onshore and offshore. Given that the broad material factors are favourable to durable and economic structures, there is still clearly a need to consider carefully all the details affecting the key life cycle stages, in order to ensure the realisation of maximum economy and competitiveness with alternative materials.

1.7 Aim of this document

The aim of this study is to encourage the development of concrete wind tower solutions and to illustrate how the benefits of concrete construction can be realised more fully by the wind industry. By focusing on key issues pertaining to wind tower fabrication, the intention of the document is not to propose definitive solutions, but rather to highlight practical methods and technologies that, through optimisation, could lead to competitive solutions.

Conceptual configurations are presented for both onshore and offshore facilities, along with design philosophies and construction methodologies for concrete wind tower solutions. These concepts have been arrived at independently by the authors of the report and, perhaps not surprisingly, there are some similarities with the thinking of others working in this field. Whole life issues are accounted for, including fabrication, transportation, installation, maintenance, decommissioning, removal and disposal. Outline solutions for both concrete and steel are used for the assessment of their relative merits and viability.

STUDY OF CONCRETE TOWERS FOR ONSHORE WIND FARMS

The onshore study was undertaken subsequent to the work on offshore wind farms. Some of the concepts touched on in the offshore study have been developed in more detail. There is inevitably some repetition between the studies, readers are advised to read both in order to get the fullest information from this report.

2 DESIGN PHILOSOPHY

2.1 General approach and configuration

The overall structural form selected for this study is a tapered tube, which has become the predominant form for wind towers over the past decade. This might be more fully characterised as a smoothly tapering, solid walled, circular section tube which can provide the following properties easily by virtue of its form:

- The weight and strength profiles of the cross-section follow the general pattern of that required by the function. Due to the taper, the weight per metre height reduces with increasing height, giving more favourable dynamic characteristics, as does cross-sectional strength, which matches the general pattern of overturning and shear forces.
- The circular form is efficient for a structure which can generally be loaded fully from any compass direction.
- Good appearance (particularly with careful attention to taper and proportions).
- Good aerodynamics.
- Provides sheltered internal space for vertical access for maintenance and cabling.
- Minimises surface area and edges open to aggressive weathering and corrosion.
- Is very well suited to concrete technology.

The choice of form may seem obvious but it is perhaps worth recalling that there is a long history of tall lattice towers for various purposes, including the traditional agricultural wind pump. In recent years lattice towers have been used in China for modern wind turbines. Some early consideration has been given to lattice or perforated tube structures in these studies as they might conceivably offer some benefits, particularly in the upper middle and top zones of the tower. However these have not been explored further.

Within the general constraints of the selected tapered tubular form, the key area of the design philosophy is flexibility of the solution. Each wind farm site is different and each project will have its own set of constraints and problems. Flexibility is thus needed at various levels of design, from overall configuration through to main strength and stiffness design, right down to key details.

At the global structural level, flexibility can be provided by a framework of related and alternative structural configurations. By fitting into the overall outline envelope of the tower, this will allow individual project designs to be more closely tailored to the circumstances of the project and thereby facilitate more cost-effective outcomes.

As shown in Figures 1.1 to 1.4, the approach involves notionally splitting the tower into three broad zones:

- Upper zone
- Middle zone
- Base zone

These are chosen to mirror different zones of design and construction constraints over the height of the tower. The extent of the zones is not fixed, so that the design concept has a sufficient degree of flexibility to cope with the changing constraints and design drivers of future wind farms.

One or more construction types for each of the zones are considered (in the following sections). Typically these are:

- The upper zone – *Precast concrete rings capped by a steel fabrication comprising the turbine yaw ring/nacelle bearing platform within a tapered steel tubular section*
- The middle zone - *Precast concrete segmented or monolithic rings*
- The base zone - *In-situ concrete or precast concrete segmented rings*

Alternatively, for in-situ slipform construction over the full height of the tower, the zoning has less apparent significance, but may still provide a useful way of characterising the different blends of design drivers over the height of the tower.

This framework is intended to facilitate practical conceptual design strategies. It may also point towards a structured approach to maintaining significant flexibility through the project preparation and procurement process for specific wind farm developments. The aim is to allow major component parts of the design to be readily adjusted during the project development programme to meet the changing market conditions. These take account of such factors as availability of materials, construction plant, fabrication facilities, contractors' preferences and risk perceptions, and possibly fine tuning to emerging load information relating to adjustments in turbine specification.

Given the major programme of wind farm development projected for the next decade and the implied steep rise in demand for fabrication and installation resources, flexibility is a desirable characteristic.

2.2 Design for construction

This section might also be called 'design for production', where construction or production encompasses fabrication, assembly and erection. The concept implies a design approach that involves significantly greater than normal attention being given to shaping the design of the final product to take account of these aspects of the construction process. It also implies that some design of the construction process itself is also involved. It is meant to put emphasis on an approach that can achieve significant process improvement and overall economy, particularly where there are potential economies of scale in terms of the numbers of units to be produced within particular projects or in the market at large. This opportunity is surely there, given that many currently planned wind farms may contain 30 to 150 units and the total future market implies the construction of many hundreds of units. It is perhaps helpful to draw attention to this concept for the general construction industry, where so many projects are essentially prototypical, although paradoxically by the nature of its business the precast concrete industry is very used to the mass production of standardised components.

In its broadest sense the concept involves an interactive and challenging approach to the values put on other design criteria. For instance, in relation to particular functional performance, it might involve critical examination of the trade-off of some loss of performance against production economy. Such an approach can usually often lead to better outcomes all round, particularly if it is applied to all levels of the design, from the overall configuration of the tower and major component segments through to all the key details.

2.3 Design concepts

The various overall configurations and design concepts for towers and details illustrated in this report have been developed in the light of the above design philosophy. They are discussed further in Section 3. Illustrations shown are indicative only and would require significant further engineering attention for realisation.

3 DESIGN

3.1 Indicative designs

Indicative designs and details for onshore concrete towers are illustrated in Figures 1.1 to 1.10. The two configurations selected for this study as representative of current and near-future scenarios are:

- 70m high tower; 2MW Turbine; 39m blades/80m rotor – Figure 1.1
- 100m high tower; 4.5MW Turbine; 58.5m blades/120m rotor – Figure 1.2

The near-future scenario is intended to cover the trend towards taller towers and/or larger turbines already well underway on some European onshore sites, where towers of 100m plus are considered to be cost-effective because of the better wind climate and therefore productivity. Interestingly the transition in height between current and future scenarios appears to require a step-change for steel towers in fabrication method and technology, and approach to transportation. Overcoming these constraints implies some relative increase in the cost of steel towers against concrete towers and therefore appears to shift in favour of concrete towers.

The figures indicate the type of concrete construction that is considered to be most appropriate for the different zones. These are:

- Upper and middle zone: Precast prestressed concrete
- Base zone: In-situ concrete, precast concrete or hybrid of the two
- Foundation zone: In-situ concrete

There is also a strong case for in-situ construction of the whole tower using the slipform method. Slipform is a continuous casting technique in which formwork is steadily moved upwards on screw jacks at a rate which allows the concrete placed in the top of the form to set sufficiently to support itself before emerging from the bottom of the rising form.

The overall design approach is to achieve best overall economy by matching construction process carefully to the particular needs and characteristics of each zone.

The taper profile of the tower is kept simple. For the 70m tower a uniform taper has been considered; for the 100m tower a bi-linear taper, with a more marked taper in the base zone giving a better fit to the strength and stiffness requirements. It is possible to achieve an even better fit by using a multi-linear taper, which does not necessarily introduce significantly greater production complexity if the taper within the height of the individual precast units is kept uniform.

A 'zonal' definition of the tower structure is given earlier in Section 2. From this perspective the idea of matching the type of construction to different zones points to the possible use of other materials for certain zones, in conjunction with concrete for others. One such hybrid configuration is shown in Figure 1.3 where the combination includes:

- Foundation, base and middle zones – *concrete*
- Upper zone – *steel*

The rationale for this configuration might be that, in the circumstances where a full concrete tower was competitive, it could provide a cost-effective way of tuning the dynamics of the tower (where weight reduction at the upper level is likely to prove beneficial in this respect). It might also provide a way of utilising available fabrication and erection resources to best effect, particularly for a major wind farm development where the pressure on a particular resource might become critical to construction programming.

Additionally this configuration provides a possible way of extending the height of present steel tower designs, without incurring the greater fabrication and transportation difficulties that are likely to arise from an all-steel tower design.

3.2 Foundations

Foundations for onshore wind towers will generally involve the use of a significant amount of reinforced concrete. Typical foundation arrangements for different soil conditions are shown below.

SOIL CONDITIONS	FOUNDATION TYPE
Good rock close to ground surface	Reinforced concrete (RC) base with rock anchors or bolts
Firm ground and underlying soils	a) RC gravity base acting as a spread footing
	b) RC base with piles/tension anchors
Weak or loose soils to depth	RC base with piles

Bases will generally be circular or polygonal (hexagonal/octagonal) in plan shape. Piles may be steel or concrete. The stiffness and dynamic behaviour of the foundation system is likely to be of concern, particularly in weak and loose soils.

Indicative weights of tower and nacelle in relation to the foundations are shown below.

WEIGHT in Tonnes	CONCRETE TOWER		STEEL TOWER	
	70m High 2MW	100m High 4.5MW	70m High 2MW	100m High 4.5MW
Tower Head Mass (THM) Inc. Rotor, Nacelle –gear box, generator etc.	105	220	105	220
Tower stem	450	1050	135	240
Foundation Allowance for base and piles	1400	3000	1500	3100
TOTAL WEIGHT	1955	4270	1740	3560

These figures are approximate and can be influenced by many variables between sites and systems. Foundation design is strongly driven by the large overturning forces that are fundamental to this type of Wind Energy Converter (WEC) system. In this respect the additional dead weight of a concrete tower as compared to a steel tower is likely to prove beneficial for foundation design in counteracting the vertical tension resultant.

3.3 Tower elements

The tower shaft may be split into a number of different sections for the purposes of strength and stiffness design, fabrication and erection. The possible types of construction featured in this study and their likely zonal application have been indicated in Section 3.1. This shows how various combinations of in-situ, precast, and hybrid concrete/steel construction might be applied, thereby allowing optimisation for different physical and development circumstances.

The use of precast concrete units for all three zones of the main shaft is an important option. A possible outline arrangement for such units is shown in Figures 1.4 and 1.5 for 70m and 100m high towers respectively.

The units may comprise a number of segments (2, 3 or more) to complete a ring, or whole ring units (cast as one unit) according to the ring section diameter and to the weight of individual units, all to take account of ease of transportation and handling. The wall thickness of units/ring sections may be varied between different zones and possibly within the separate zones. Similarly, concrete strength may be varied to achieve an optimum balance between the various cost drivers and physical constraints.

The construction sequence for an all-precast tower as shown in Figure 1.6 is described further in Section 3.7.

3.3.1 Base zone

Generally in-situ concrete is required for foundation construction (Section 3.2). The construction of the base zone of the tower using in-situ concrete would not require the deployment of substantially different resources, apart from the particular formwork system for the base zone of the tower shell. Wall thicknesses typically in the range of 350-400mm plus would be vertically prestressed. This wall thickness is needed for strength. Thickening the wall even further in this zone may provide an efficient way of tuning up the overall tower stiffness since, amongst other things, the additional mass here has very little effect on the natural frequency of the structure and can in fact provide a useful added compressive load on the foundation.

Prestressing tendons could be accommodated within the wall thickness if desired and as such could be either bonded or unbonded. The overall construction operations have quite similar logistics and do not incur heavy additional costs in this respect.

Alternatively, the use of thick walled (300-400mm) precast concrete segments can provide a solution. These would be conditioned in size for transport to the site, and might have a width of 3-3.5m and a length (height when erected) of 12-18m. They would probably be best erected in-place before joining with in-situ poured concrete stitches to form a monolithic ring shell. Such joints could be conveniently made as wide joints, with projecting reinforcement from the precast units interlocking within the joints.

In either case the base zone construction in concrete can easily accommodate the local details (doorway etc.) for access to the tower. The design can be easily adapted to provide the base support to an upper steel tower as a hybrid solution.

A hybrid design may offer an immediately attractive solution for those projects with towers above 80-90m height, which are already essentially committed to the use of steel towers. Such an approach could avoid the step change in steel plate thickness required for the shell in the base zone, and thereby avoid the additional fabrication and transportation difficulties that may be experienced with the use of the heavy large diameter steel tower shells for such towers.

3.3.2 Middle zone

Weight minimisation of the middle zone upwards is key to achieving a cost-competitive solution for a full-height concrete tower design.

For a given outline profile of the tower, the direct variables affecting the weight of the tower shell are effective concrete density (i.e. the density of the concrete reinforcement composite) and wall thickness. For the present study the use of lightweight concrete (using special manufactured aggregates) has not been considered in any depth. The unit weight (density) of concrete considered for structural design purposes is relatively independent of strength and for present purposes has been taken as 24 kN/m^3, which is typically used for reinforced concrete with moderate levels of steel bar reinforcement.

The wall thickness is taken to be uniform for individual units (except locally at prestressing tendon anchorage points on certain units only). Various drivers include structural strength, stiffness, local stability and durability requirements which define minimum concrete cover to steel reinforcements or otherwise unprotected steel prestressing ducts and strand.

In the middle zone upwards the minimum wall thickness may be determined by the ‘concrete cover to reinforcement’ requirement rather than by the necessary strength and stiffness. This becomes more critical as the design strength class of the concrete is increased.

It is considered that even by using only a single centrally placed layer (two-way grid) of normal steel rebar, but taking account of realistic placement tolerance even with factory production, it will not be practicable to achieve sufficiently reliable durability for a long life tower structure (50 years plus) in an aggressive external environment with a wall thickness of less than 150 to 175mm.

Overcoming the restrictions imposed by the normal concrete cover requirement allows the use of thinner concrete walls. This could be important to the minimisation of the weight of the middle and upper zones of the tower and the cost of the tower overall.

Various techniques are available which can overcome these restrictions. These are discussed further, along with a possible design solution, in Section 3.5.

3.3.3 Upper zone

The design logic of weight minimisation could be even more relevant to the upper zone. It is envisaged that the thinnest practicable shell elements will be used here, subject of course to the provision of sufficient resistance to the maximum cyclic wind effects. For present purposes the minimum wall thickness has been set at 100mm.

The top of the tower has to support the turbine nacelle and yawing bearing. For present purposes it is considered that this connection will be provided by means of a 'turbine spur'. Notionally this will be a two metre high section of steel shell matching the out tower profile, within which there is a transfer platform supporting the yaw ring, and through which there is a central access hatchway into the nacelle. This fabrication would be fixed to the head of the concrete tower by a ring of stressed bolts.

3.4 Prestressing of the tower

The concrete tower is designed as a vertically prestressed tapering tube. A circumferentially uniform pattern of prestressing forces is to be applied at a magnitude which changes with height (in a stepped pattern) according to the forces that the tower has to resist. This will provide a circumferentially uniform vertical compressive stress sufficient to avoid vertical tensile stresses in the tube walls under normal design loading. This vertical prestress is also important to the shear strength of the tower tube.

Prestressing provides an efficient and economic means of achieving good all round strength with excellent durability, fatigue and dynamic performance. It also provides a straightforward and positive means of unifying the separate rings of a precast concrete solution so that they act as a monolithic tower structure. With careful design and detailing, prestressing can also provide solutions to the problems of handling units for erection and maintaining adequate strength in the partially completed tower during its construction. This can be further appreciated by reference to Section 3.7 and Figures 1.6 to 1.8.

The choice of general technique and configuration lies between bonded and unbonded tendons, and internal or external placement. In general, unbonded external tendons are favoured in this onshore study for their simplicity and ease of installation although bonded or unbonded internal tendons could also provide a satisfactory solution, particularly for the thicker sections of wall and especially in the base zone.

External prestressing involves the placement of the prestressing tendons (cable or bars) outside the structural section which is to be prestressed, and therefore removes the need to accommodate the tendons and any ducts within the thickness of the shell wall. The stressing force is imposed through an anchorage detail that transmits it back into the section. An anchorage is of course required at each end of the tendon. The tendon is not continuously bonded to the section in question although it may be restrained back to it at discrete points along its length. In the case of the tower, vertical prestressing of the tower shell may be applied by a circular array of tendons placed within the concrete tower shell close to its inside face.

Crucially the use of external prestressing permits the use of thin-walled concrete sections for the tower, since wall thickness is not governed by the need to accommodate tendons internally. Another important advantage is that installation of the tendons at the various stages of construction can be more easily achieved.

It might be argued that the external prestressing tendons will be more at risk from corrosion. There is long experience of the use of external tendons on major bridge structures. Whilst care always has to be taken to ensure the durability of highly stressed components, particularly in the anchorage areas, there is no reason to think that this represents an unusual or insolvable problem. The tendons would be protected by factory installed corrosion protection systems, including inhibitors, greasing and heavy plastic sheathing. Anchorage areas will be accessible for proper finishing, capping and inhibiting grease injection. Additionally, unlike internally located prestressing strands, the external tendons would be visible, easily inspected and even replaceable. Location within the internal tower shaft provides a sheltered and relatively benign environment within which humidity and condensation could be actively and economically controlled as a further precaution.

There are further advantages from using external unbonded prestressing. There will be even higher fatigue tolerance of dynamic loading because the cables will experience an overall averaged and therefore lower stress range during load cycling, as compared with bonded cables which are subjected to rather higher more localized stresses. There will also be lower friction losses during stressing. The eventual demolition of the structure will be simplified, and the recovery of both the concrete and steel component materials for recycling will be simpler and cheaper.

Outline details for cable layout, anchorage and restraint are shown in Figures 1.6 and 1.8.

It is considered that some restraint of the individual unbonded tendons is required, linking them back to the tower shell. Figure 1.10 shows a simple detail that would provide such restraint. This restraint would provide some additional stability to the tower under accidental overload or high level impact as might occur during erection. It would also act as a safety measure during construction by restraining any whiplash movement should a tendon inadvertently fail due to overstressing during the stressing operation.

3.5 Thin walled shell units

The importance of minimising wall thickness to the economy of the concrete tower design has been discussed in Sections 3.3.2-3. This is particularly important when cranage is needed to lift large units into place. For a vertical direct load and /or overturning bending capacity, the tube wall thickness can be reduced by increasing the concrete strength class (it is considered practicable to increase this up to a compressive strength of 100N/mm^2 as required, and possibly beyond).

However there are limitations to the minimum thickness of conventionally reinforced concrete sections due to the need to provide a minimum thickness of concrete cover as explained in 3.3.2 and possibly from the accommodation of prestressing tendons. An approach to overcoming the latter limitation has been set out in Section 3.4. The concrete cover restriction may be overcome by the use of non-corroding reinforcement.

Some possible techniques and ways that these might be used are explained below:

i) The use of non-corroding reinforcement would remove the need for thick cover (50-75mm) required for external or aggressive environments, and would allow cover thickness to be reduced to dimensions necessary to allow good penetration and compaction of the concrete around the reinforcement, and provide good bond strength and transfer of forces from the reinforcement into the body of the concrete. By limiting rebar diameters to small to medium range (10-25mm dia) cover dimensions could be reduced to 25mm assuming that the durability issue had been overcome.

   Assuming the use of steel rebar, the options are epoxy coated, galvanized or stainless steel rebar. Both the coated bars (epoxy/galvanized) are susceptible to mechanical damage or local weakness of coating, and to consequential anodic corrosion which could limit their durability and reliability for long life structures. For the present study it is assumed that stainless steel rebar will be used where steel reinforcement is required in thin wall situations, or in other details where low cover is desirable or necessary for design reasons. It is generally accepted that stainless steel rebar can be used in conjunction and in contact with normal steel reinforcement within concrete members without any serious risk of 'cathodic/bi-metallic' corrosion problems.

   It is important to note that stainless steel costs approximately five times more, weight for weight, than normal high tensile steel rebar. It is therefore vital to limit the use of such reinforcement to a minimum in order to achieve the potential economy of thinner walled sections.

ii) Other techniques for achieving non-corroding reinforcement include the use of non-ferrous reinforcement in an analogous way to conventional steel reinforcement or by using significant quantities of chopped fibre as part of the concrete matrix. The former approach has not been considered any further here as such reinforcements are not in widespread current use with concrete and do not appear to offer significant overall advantages at present.

   On the other hand the use of chopped fibre reinforcement concrete (FRC) is being widely used in various applications. Techniques and supply arrangements are well established with a significant international research effort continuing to achieve improved structural properties and performance for FRC. Suitable chopped fibre materials are certain polymers (polyesters/polypropylenes), stainless steel wire or arguably even normal steel wire. Fibre reinforcement can give considerably increased resistance to cracking and much greater toughness over plain concrete. At present FRC cannot be relied upon to provide reliable tensile strength to resist major primary structural actions.

The question arises as to what strength characteristics are needed for the concrete tower shell? The tubular form provides excellent vertical load carrying capacity and there is no primary need for tensile reinforcement for this purpose. Overturning bending of the tower will be resisted by the action of the prestressed concrete tube, as will horizontal shear and torsion forces about the vertical axis. Bending in the wall shell about the vertical axis and associated shear forces, and horizontal hoop forces, are generally likely to be very small from externally imposed forces. Such forces, to the extent that they are significant, are likely to be the result of strain induced loads from temperature, creep etc. It is considered that these secondary effects can be resisted safely by local reinforcement and the provision of a level of general toughness and crack resistance.

The effects of loadings from the turbine at the very top of the tower, particularly cyclic loadings, may require wall thicknesses greater than the minimum achievable. However moving down and away from the uppermost level, the stress levels from these loads will drop with the increasing section diameter, and may provide a zone of tower where these walled units provide an appropriate economical and useful design solution.

By combining the use of limited quantities of stainless steel reinforcement with the use of FRC it is considered that thin-walled shell units with the necessary structural properties can be economically constructed.

Fibre reinforcement is proposed as the most suitable material for alternative concrete reinforcement. Fibre reinforcement is to be used throughout the precast ring segments, but with all the steel reinforcement omitted over the central region

of the rings. The prestressed design of the concrete tower ensures that for ultimate limit states, no tension occurs in the concrete. Consequently the fibre reinforcement provides some general toughness and resistance to secondary effects such as thermal stresses. Bands of stainless steel reinforced concrete are located at the ends of each ring segment to provide extra toughness during lifting and handling and act as 'crack stops' down the tower. Figures 1.7 – 1.10 show the arrangement for a typical ring segment.

It must be emphasised that while this approach uses existing concrete technology to its limits, in essence nothing new is being proposed. The materials are well proven for the functions they perform. However, this approach could play an important part in achieving a precast concrete solution that can be both cost competitive and able to challenge the current steel solution.

3.6 Joints

These comments apply to the precast concrete solution or possibly to the joint between an in-situ concrete base and a precast middle/ upper section of tower.

Simplicity of connection and accuracy (and trueness) of level are the two key drivers for joint design. The simplicity of the connection will keep the formwork costs and on-site construction time to a minimum. Accuracy of level across the top of the joints is imperative in order to ensure verticality of the final complete tower.

Two basic types of joint are envisaged. One will be at a connection of the precast units where the prestressing strands terminate, the other will be at an intermediate connection of the precast units where the prestressing tendons are possibly linked to the tower wall but not terminated.

The joints will be provided with a simple mechanical connection such as stainless steel dowels to facilitate accurate engagement and assembly, to fix each unit's location immediately after placement and before overall prestress is applied. This takes account of the possibility that prestressing may not be applied until several units have been stacked up.

One possible concept for achieving a self-levelling joint involves the use of three local seating pads on each side of the joint to ensure verticality to within a few millimetres. The seating pads are envisaged as thin projections above the general joint surface that would be ground to a precise level to set the general thickness of the joint. The thin joint will be filled with epoxy resin. The method of application of the resin would need to be carefully developed to be weather tolerant and reliable. It may involve such techniques as injection or pre-placed mortar/ resin pre-placed impregnated tapes. Some of the stainless steel dowels would be made longer to act as locators to facilitate rapid acquisition and correct orientation of the incoming unit, and achieve fast connection of the joints in the difficult conditions on site.

It is important that simple jointing and connection methods are devised to facilitate straightforward and rapid erection. Indicative details are illustrated in Figures 1.7–1.10. These details are based on the use as far as possible of plain thin epoxy joints between units, with the units coupled vertically both individually and overall by prestressed bolting and tendons.

3.7 Construction sequence

A notional construction sequence for a precast concrete tower is illustrated in Figure 1.6.

The precast units will be transported to site as whole rings for smaller diameter sections and as ring segments for the larger diameter rings. Ring segments will be joined into a complete ring on site. Ring sections will be stacked up, either in place for the lowest heaviest rings, or into tower sections comprising three or four rings for lifting into place on the tower. Some limited stressing together of tower sections prior to lifting into place may be useful to assist with handling, or alternatively a special lifting cradle may provide a quicker and more cost-effective solution.

Newly erected sections of the tower would be stressed either by bolts/tendons connected to the underlying section, or by full length tendons emanating from the base anchorage. This would provide both the short term stability during construction and contribute to the permanent works strength. The aim would be to achieve an efficient pattern of curtailment of tendons and consequently applied prestress which was well matched to the design envelope of externally applied force resultants.

This approach to prestressing requires anchorage arrangements at a number of levels over the height of the tower. Figure 1.8 indicates a possible outline detail involving the provision of an annular ring beam formed inside the tower shell on selected units. The ring beam would contain vertical duct holes at regular spacing around the perimeter set to align with the planned tendon paths. This detail could provide anchorage to long tendons, short bolts, or act as a local duct for tendons continuing up the tower. The ring beam would require relatively heavy local reinforcement in accordance with normal practice for prestressing anchorages. This would probably be best provided by stainless steel bars in the form of welded cages, allowing cover to be minimised and thereby reducing the propensity for cracking and also of course minimising the additional weight of the ring beam.

The number of tendons, curtailment pattern, anchorage positioning, and so on are interlinked with the requirements for erection and construction as well as those for the permanent design. Careful design and planning at all levels of detail will be an essential part of producing an efficient and cost-competitive solution.

3.8 In-situ slipform construction

Precast concrete units offer a viable design solution for the whole tower, and also for the middle and top zones in combination with an in-situ concrete, slipformed base zone. Alternatively, in-situ slipformed construction for the entire pylon height offers some potential advantages, avoiding the need for transporting precast units to the wind farm site and the provision of heavy cranage. This could be particularly useful for sites in areas where access for very large vehicles is problematic. Whilst in-situ operations clearly involve the transportation of basic materials to site, this can be undertaken off critical timelines and does not pose special problems of vehicle size or load size. State-of-the-art mixing equipment such as volumetric truck mixers or mobile site plant can be easily used to deliver high quality concrete on site.

Minimum wall thicknesses, achievable without incurring special difficulties, are in the range of 150–175mm. Below this thickness, proneness to horizontal cracks due to slipform drag may become a problem. Certain minimum levels of steel reinforcement are also required to provide the necessary early support and strength. Thus the 'thin shell' concepts described in Section 3.5 do not apply to the same extent to in-situ slipforming. In general this is not likely to be a significant source of diseconomy, since cost and productivity considerations of heavy cranage do not apply to slipforming.

With regard to productivity and programme time, slipforming, by its nature, is a continuous 24/7 operation with output rates of four to six metre heights per day routinely achievable.

3.9 Cranage and lifting

A current feature of wind farm construction is the deployment of heavy long jib cranes to handle and assemble the large and heavy components or units. This may involve lifting from the transporter for preassembling of sub-units on the ground, and then lifting assembled units from the ground, or lifting direct from the transporter into place on the foundation or partly constructed tower. Such cranage is expensive, particularly if there are delays due to weather, supply or assembly problems.

Slipforming illustrates a technique which removes the need for cranage, at least for the tower construction itself. At present it would nevertheless be necessary to deploy a heavy crane for lifting the yaw ring/ nacelle/ generator and the turbine rotor either completely separately, or in a 'bunny ears' configuration with the nacelle.

A fundamental feature of slipformed tower construction is the use of the existing tower to provide the main load path to ground for the raising of the new construction. The tower also provides the main stabilizer/ lateral strength for the man/ materials lift mast. This approach, using the partly constructed tower as a major component of the 'temporary works', provides a possible pathway to greater economy and construction simplicity for the future, whether for lifting precast concrete units, nacelles or others.

It would be interesting to speculate on the form of devices using this approach. It is perhaps useful to note that there are features of a concrete tower that would seem to give greater scope and facility for accommodating such devices, compared to a steel tower. The greater wall thickness of the concrete tower can more easily accommodate the imposition of high local wheel or clamping loads, whilst the provision of embedded fixings for track-ways or masts are also less likely to cause stress raiser or notching problems, which could adversely affect fatigue strength and durability.

3.10 Structural analysis

The ultimate strength of the tower was determined from the loading at the turbine's maximum operating wind speed. The extreme design bending moment at the base of the 70m tall tower is 70,000 kNm and for the 100m tall tower is 210,000 kNm both with a load factor of 1.5.

The dynamics of the tower were assessed by calculating the natural frequency of the tower and comparing this to the 1P-range (single blade passing frequency). The range for the first natural frequency of the towers was found to be 0.4Hz to 0.8Hz for the 70m high tower, and 0.3Hz to 0.7 Hz for the 100m high tower, encompassing the variability of soil stiffness, concrete stiffness and turbine mass likely. This is similar to other such towers for wind energy converters. The dynamic load inputs into tower structures depend upon the characteristics of both the rotor/ generator characteristics, the tower/ foundation structural system characteristics and their interaction.

Within the 'turbine' system there are many variables under the manufacturer's control, including active control/ management algorithms, which affect its behaviour of, and therefore inputs into, the overall structural system.

At general concept stage it is not possible to take full or precise account of all this complexity, although a range of representative values for the major load effects have been considered. It is believed that prestressed concrete towers, particularly in the middle/ lower tower zones, will be less sensitive than steel towers in their critical design sizing to the variations between different turbines of similar rating. Exploring and determining this sensitivity is an important part of the next stage of work in developing economic concrete designs.

There is ample scope to tune the towers' dynamic performance both during design and during commissioning. During design the Young's modulus of the concrete, the foundation and in-situ base section geometry and mass, and the mass at the top of the tower can all be varied to produce the desired dynamic performance for the tower. Once constructed the mass at the top of the tower can be altered to tune the tower based on as-built testing. This might be useful during prototype development. The use of mass damping at the top of the tower in the turbine would also allow the dynamic performance of the tower to be tuned.

This ability to tune the dynamic performance of concrete towers, including the ability to tune the tower during operation, reduces the potential risk of adverse dynamic behaviour of the tower. It ensures that the concrete tower solution can be designed to avoid interference with the 1P-range and can be tuned to individual site constraints.

An assessment of the potential fatigue damage to the tower design over a 25 year design life was also carried out, with concrete performing favourably.

4 QUANTITIES AND COSTS

The early estimation of costs of innovative design solutions is often very difficult as, by the nature of the situation, little direct data is available to the designer. It requires considerable engineering design, construction and logistics planning to get to a point where high levels of assurance can be given on cost estimates. In the case of wind energy much of the commercial and detailed construction cost information is held by the wind turbine companies and to some extent the wind farm developers. It is understandably regarded as intellectual property which is carefully guarded and, if available at all, such information is usually very general with limited contextual supporting information. In this study we have based our judgement of costs on basic and comparative cost exercises using estimated quantities of the main constituent materials coupled with unit rates adjusted for the adverse or advantageous factors that are judged appropriate.

The costing exercise was undertaken on the two demonstration all-concrete design solutions (70m tall tower and 100m tall tower). Costs were calculated based on a worked-up cost per tonne for the installed tower (excluding foundations and turbine) of around £580/te to £600/te, and using the tower weights detailed in Section 3.2. Comparisons with estimated costs for equivalent 100m and 70m tall steel tower show that the demonstration concrete tower designs are at least cost competitive, with potential cost savings of up to 30%. In this respect it is considered that slipformed towers provide the greatest potential savings. These savings are postulated on the basis of a substantial design development process coupled with the construction and testing of some prototypical towers. They also assume that reasonable economies of scale are available from the size of the individual development projects and from some continuity on key features of the design between projects.

5 ONSHORE STUDY CONCLUSIONS

- The contribution of wind energy to the UK's supply of electricity is planned to grow substantially over the next decade, with the number and size of wind farm projects in process and being commissioned now starting to rise in line with these plans. Therefore the demand for turbine towers is also rising in both numbers and size.
- The wind turbine industry is looking for better solutions for tall wind turbine towers of 100m or more in height. This is being driven partially by the imminent release of a new generation of larger wind turbines with longer blades, as well as the reduction in the availability of unexploited 'favourably windy' sites which will drive the need to exploit less windy sites, with less advantageous topography.
- 100m tall towers are at the limit of existing steel designs and the installed fabrication technology needed to produce them. They also present serious difficulties for transportation of the lower steel sections for the taller towers.
- The trends towards increased size of turbines (rating, weight), tower height and therefore diameter and weight, and to larger wind farms all provide more favourable competitive opportunities for concrete towers.
- Concrete towers (100m plus height) have been used successfully elsewhere in Europe, particularly in Germany. These have featured both precast and in-situ slipformed designs, similar in general terms to some of the design options considered in this study.
- All the designs considered in this report involve the use of vertical prestressing. Various configurations, techniques and details have been proposed, based upon the existing state of the art.
- In order to achieve sufficiently competitive full height solutions which seriously challenge onshore steel tower designs in the UK, substantial design and development effort to optimise concrete designs is needed.

- Key issues for attention are minimising material content and structural weight, particularly in the middle and upper zones of the tower, whilst achieving construction simplicity with short construction times. Concrete technology and the material itself offer great flexibility for optimising the design solution for particular circumstances.
- Useful weight and cost savings in the middle and upper zones of the tower may be available from some development in the use of fibre reinforcement, as suggested in the report.
- The use of in-situ slipform construction methods provide an efficient way of overcoming the difficulties of transporting large tower rings/sections to site, and removes the need for the use of a large crane for the tower erection.
- The development of 'craneless' methods for the erection of the turbine (rotor) nacelle and rotor using lifting platforms climbing up the tower itself could offer further important overall economies. The concrete tower shell is well suited to supporting local concentrated loads that would be imposed by such devices.
- The design philosophy presented draws on concrete's flexibility and promotes an approach to a solution that can be adapted to suit the individual demands of each wind farm development, and thereby provide a more cost-effective outcome. The use of precast concrete, in-situ concrete and steel in various composite or hybrid configurations is allowed for.
- Exploitation of the durability and longer fatigue life of prestressed concrete towers could provide greater sustainability and long term economic benefits. With the tower service life extended to between 40 and 60 years, one or more turbine re-fit cycles could take place with minimal further expenditure on the tower, foundations and other infrastructure.
- Concrete solutions for a 70m and 100m tall tower are derived from the design philosophy. They show cost competitiveness with likely equivalent steel solutions. Costings indicate that there is a potential for savings of up to 30% over steel towers for taller towers (70-120m plus) supporting larger turbines.

6 FIGURES - ONSHORE WIND TOWERS

Figure No.

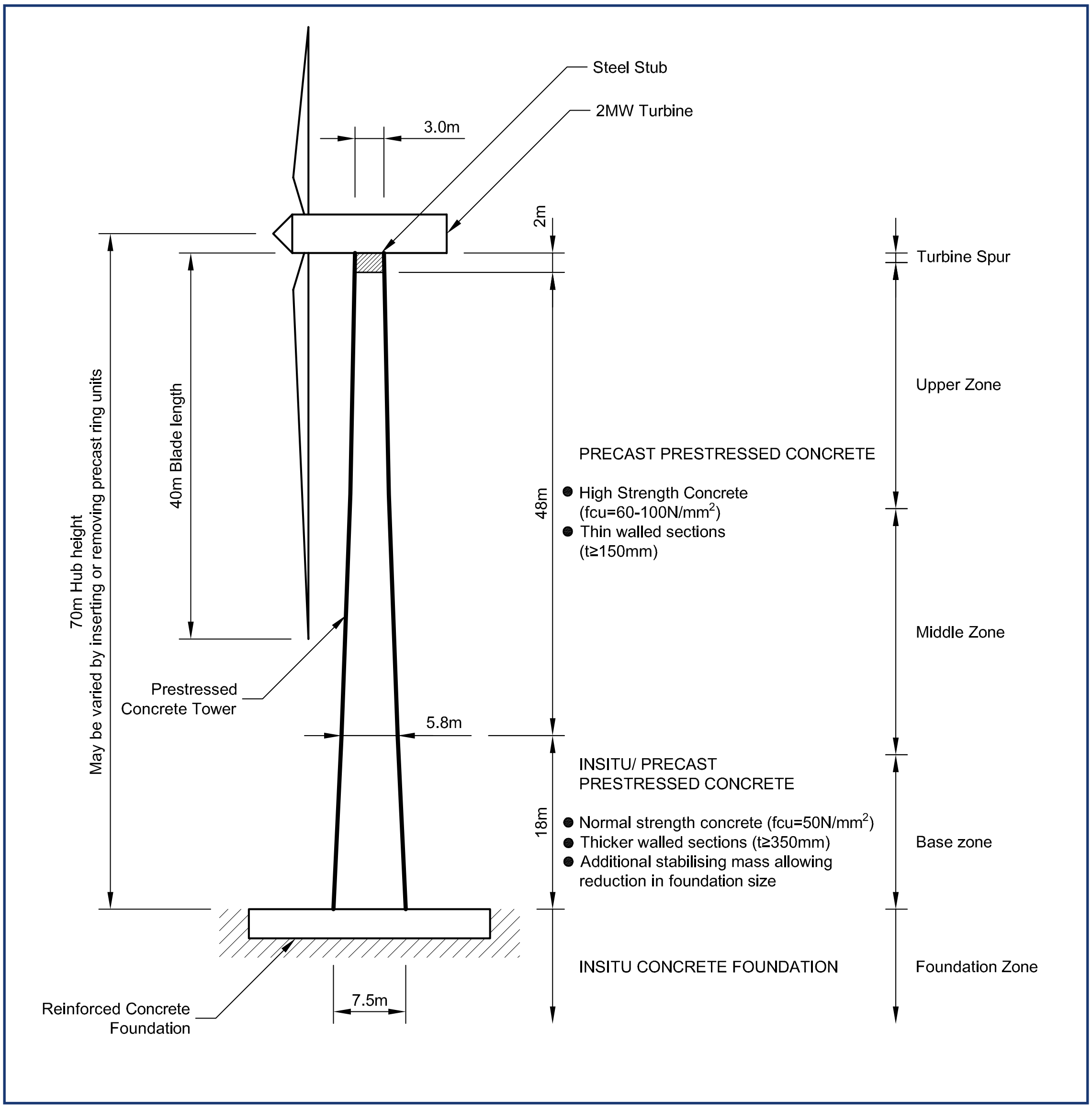


Figure 1.1
70m CONCRETE TOWER - Outline and indicative dimensions

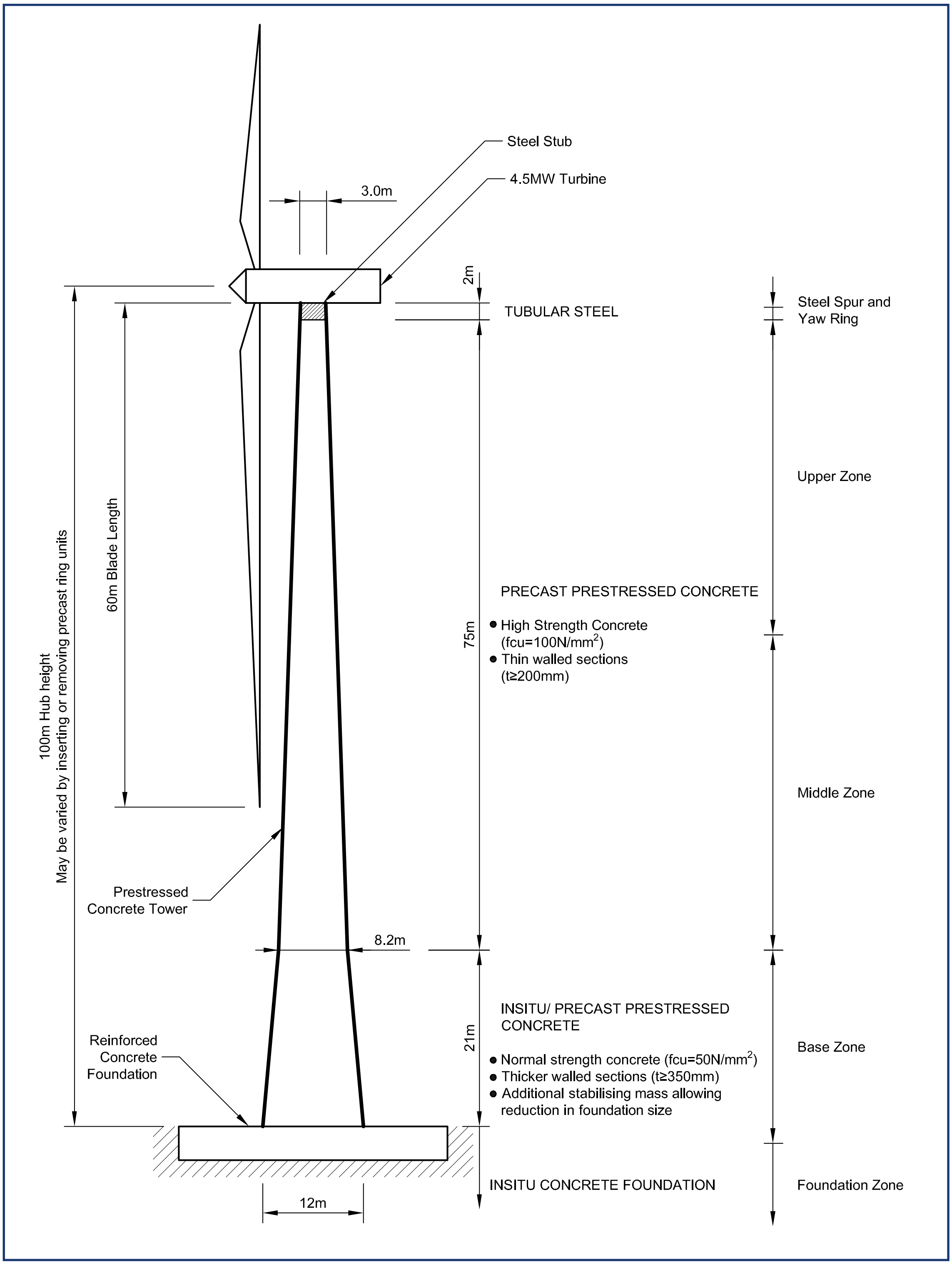


Figure 1.2
100m CONCRETE TOWER - Outline and indicative dimensions

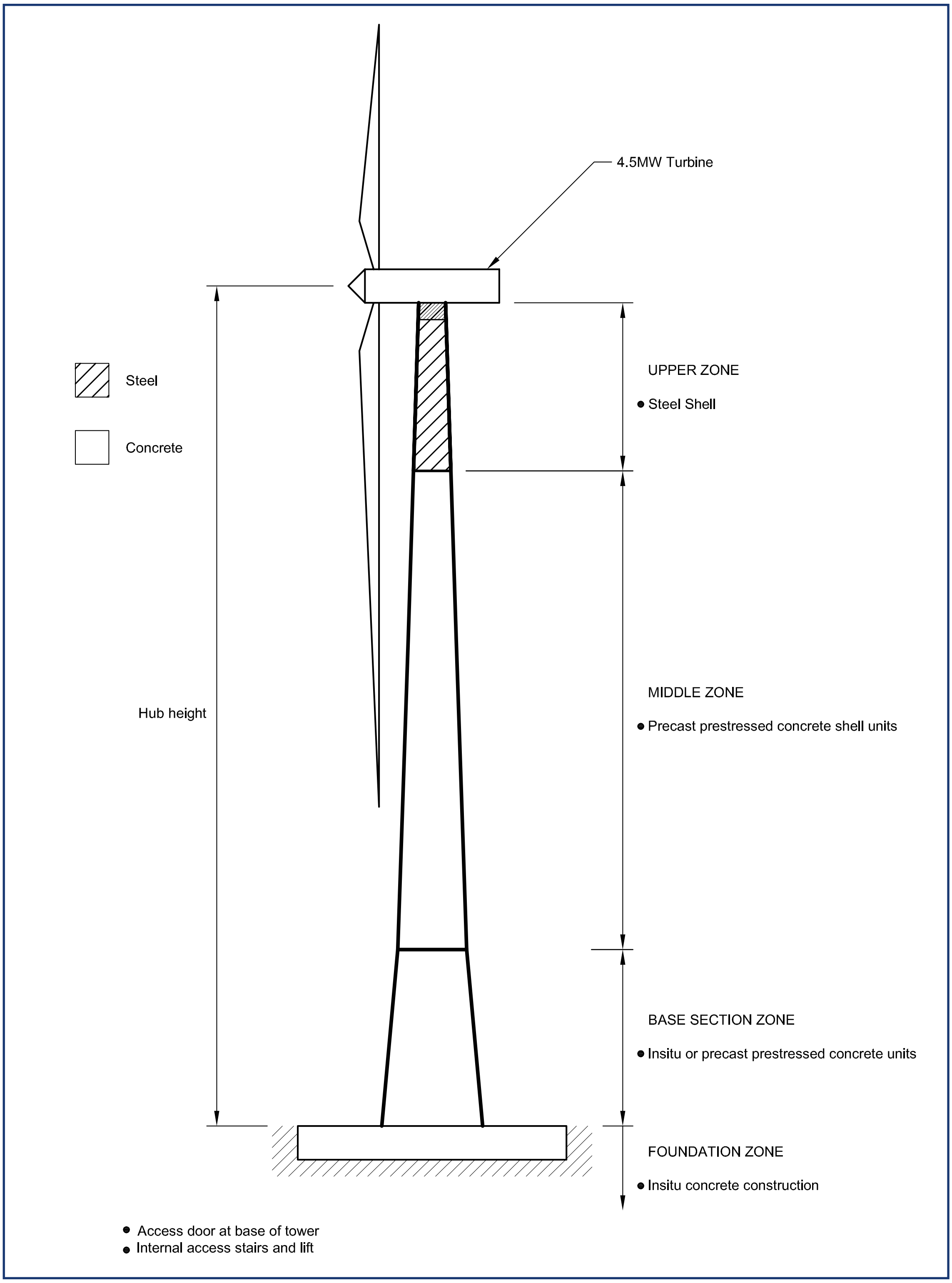


Figure 1.3
HYBRID TOWER - Arrangement with steel upper section

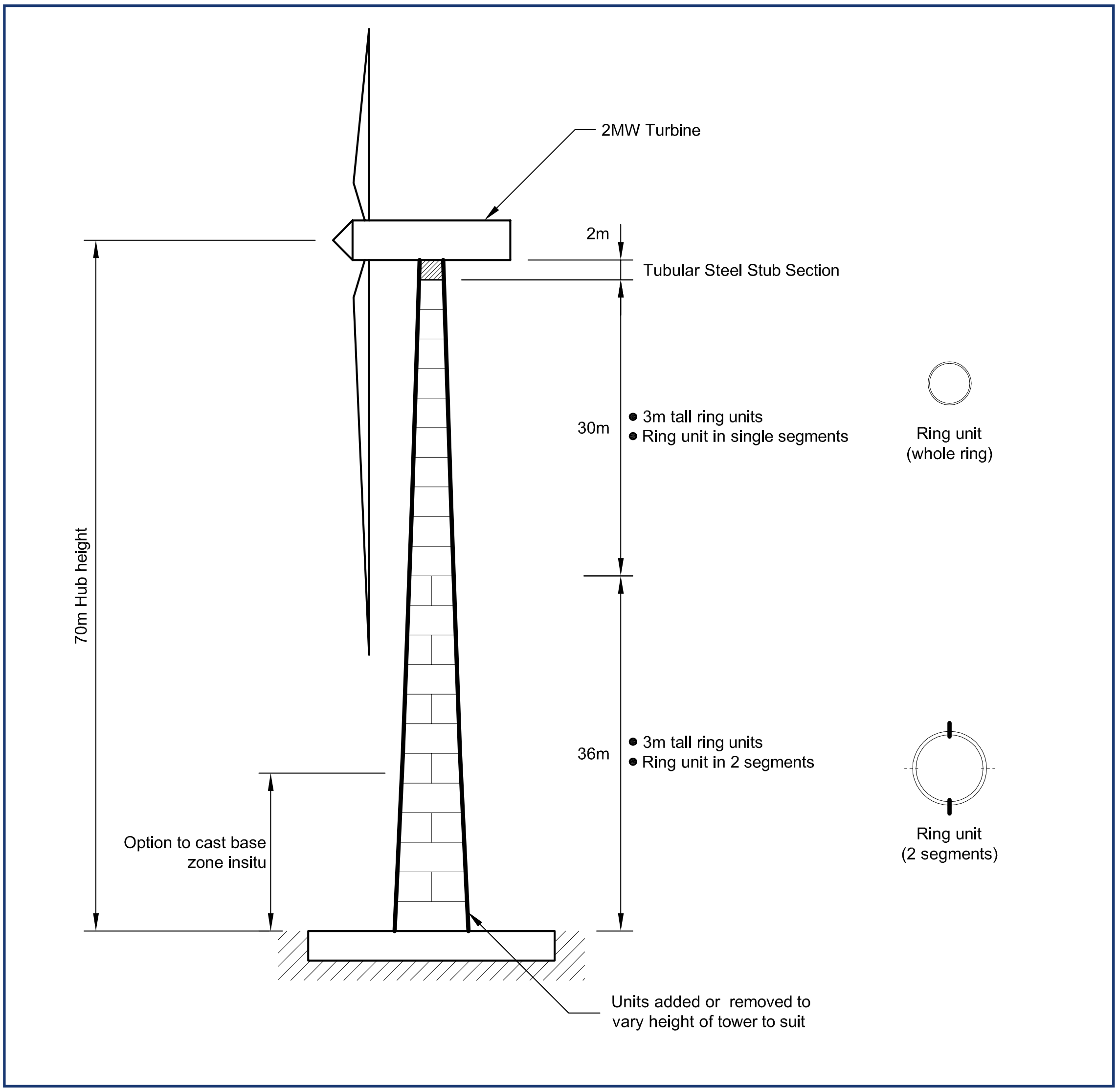


Figure 1.4
70m CONCRETE TOWER - Assembly from precast concrete units

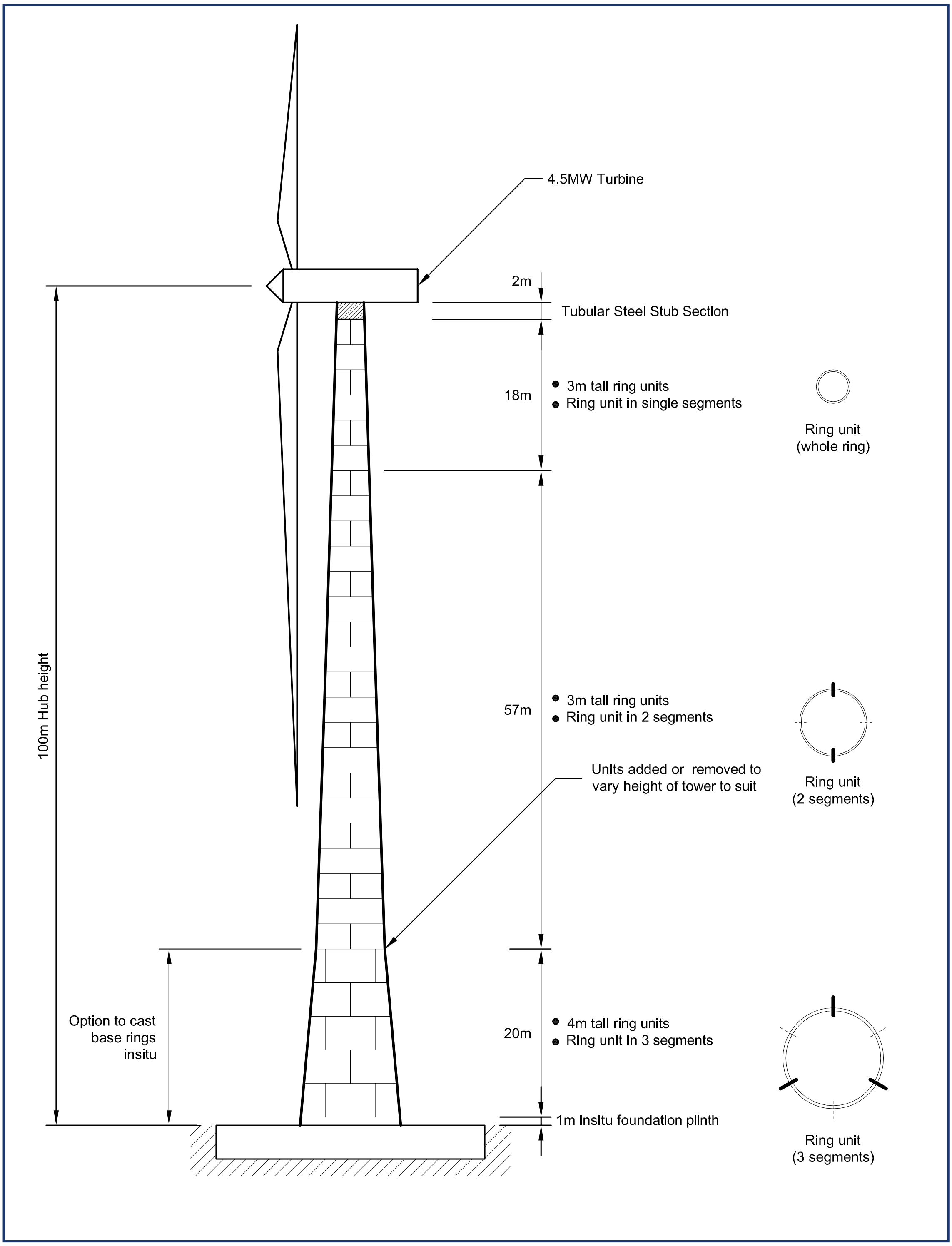


Figure 1.5
100m CONCRETE TOWER - Assembly from precast concrete units

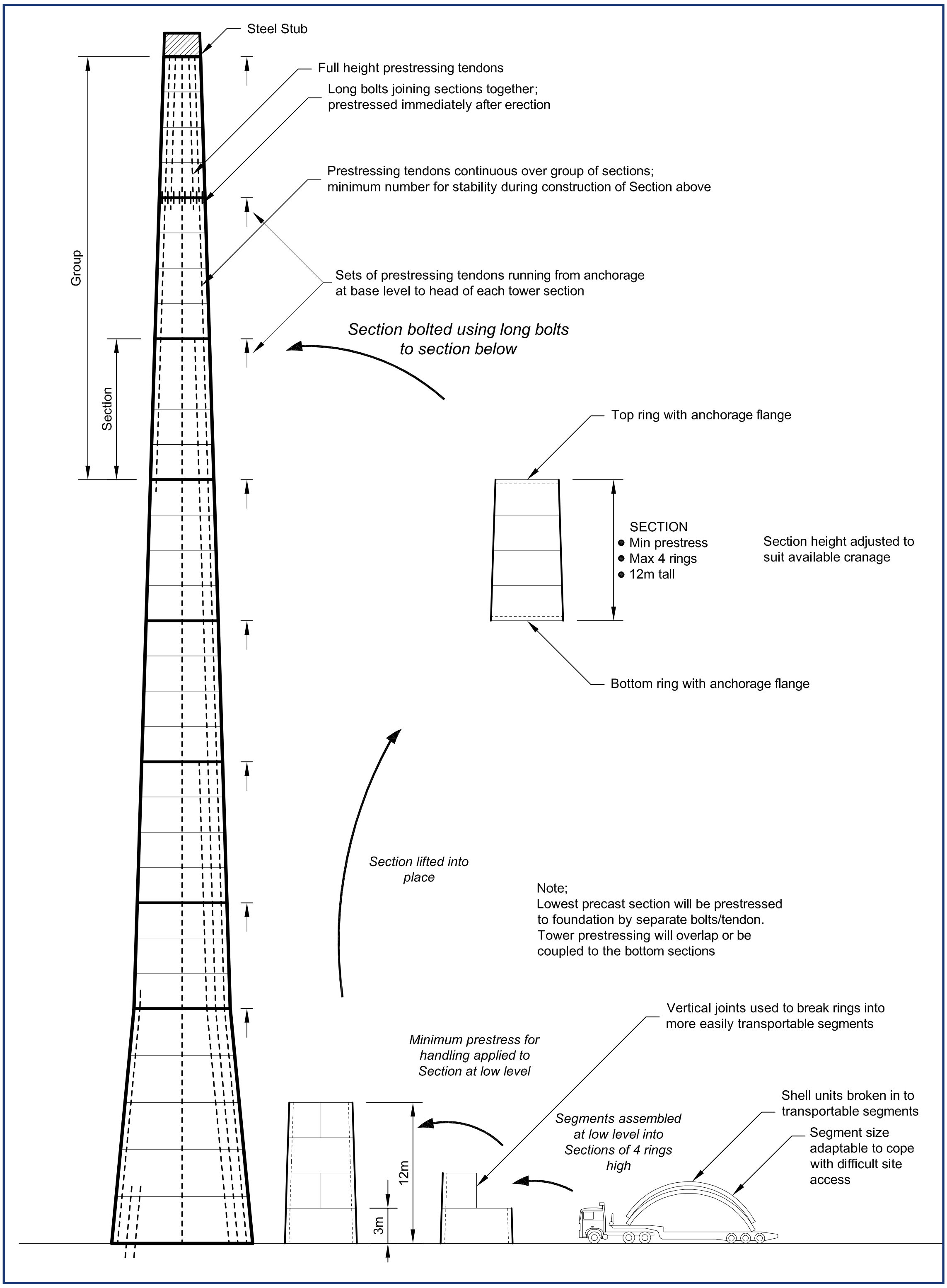


Figure 1.6
TOWER CONSTRUCTION SEQUENCE

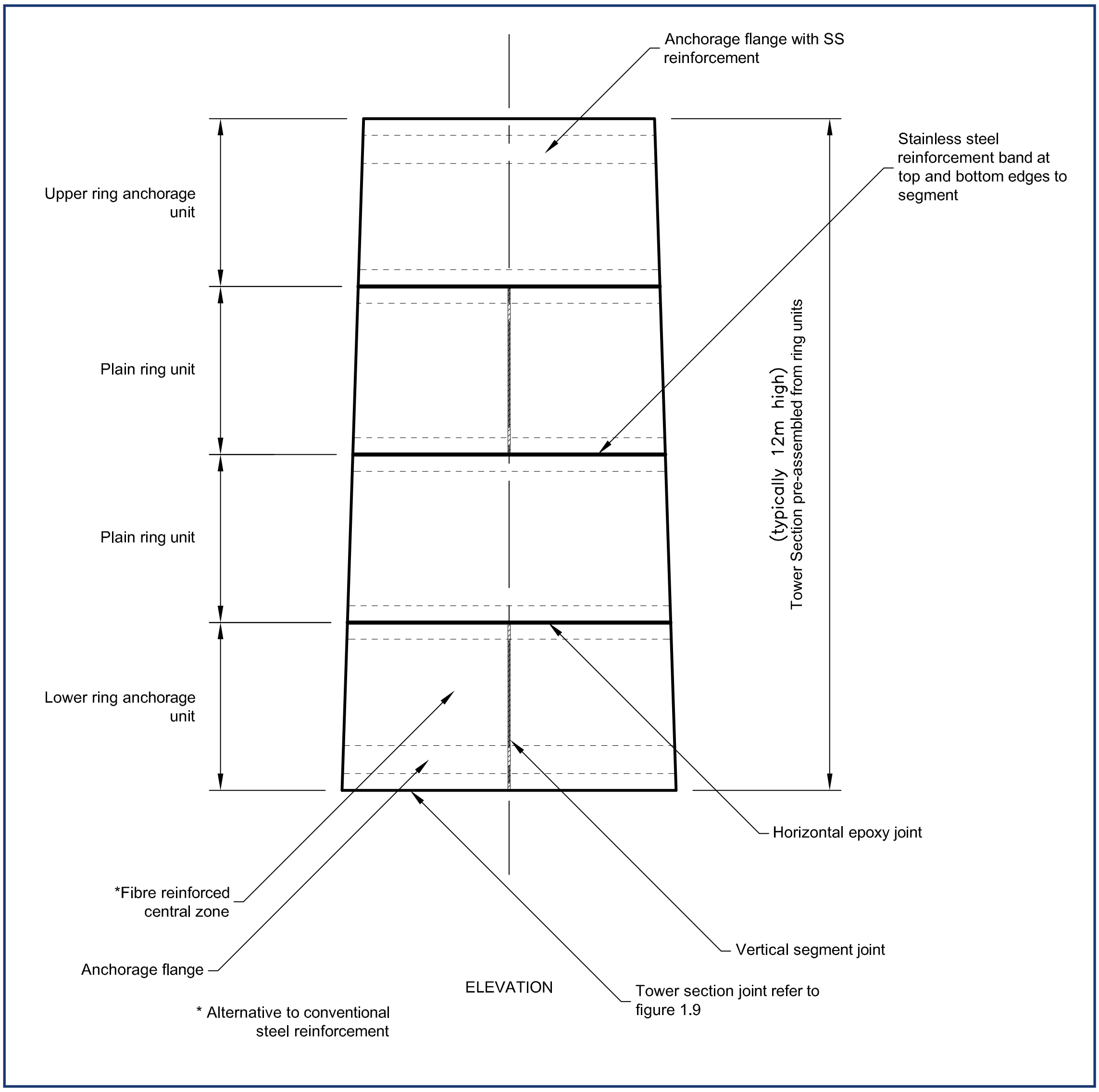


Figure 1.7
ASSEMBLY OF RING UNITS INTO TOWER SECTIONS

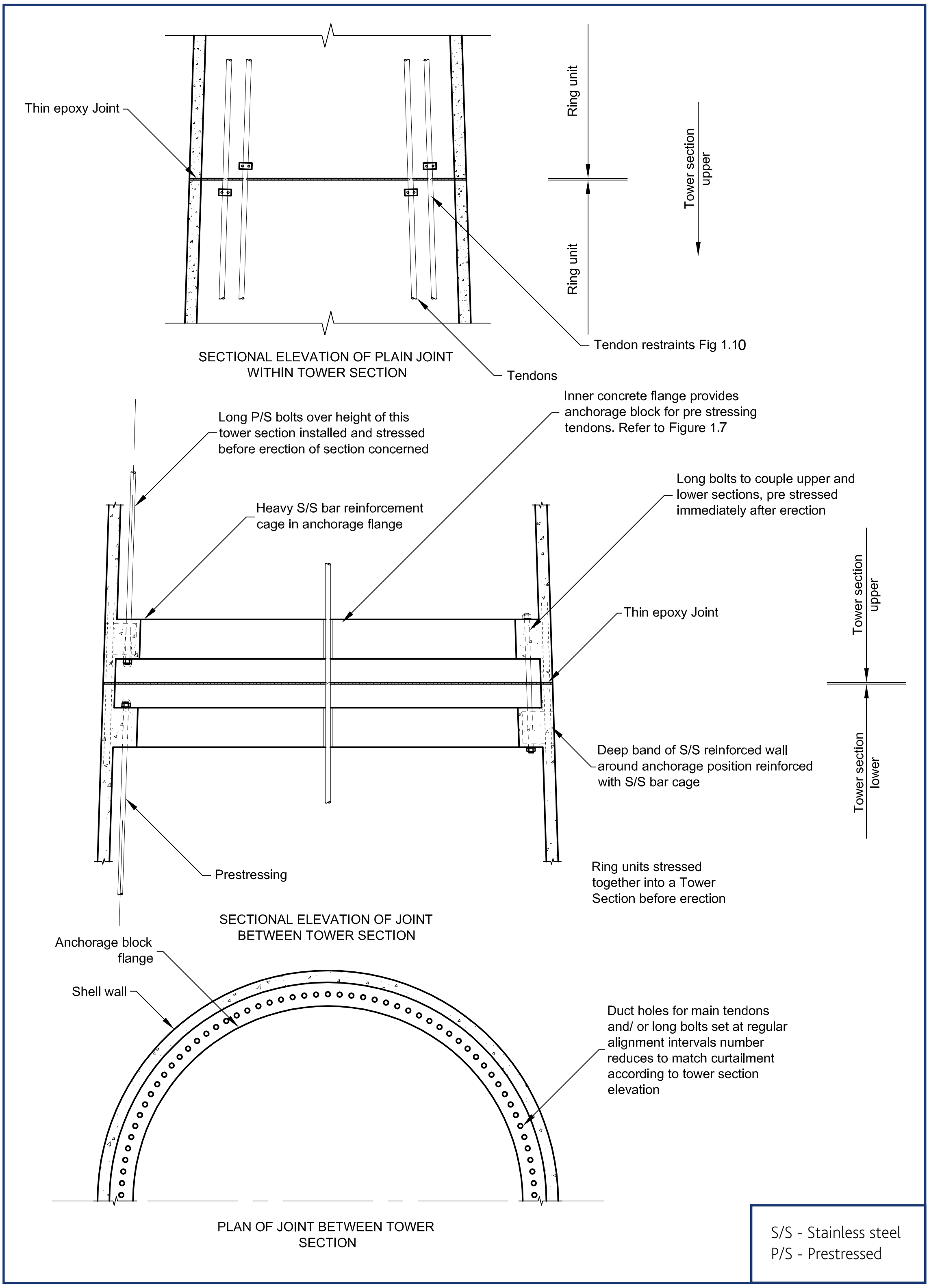


Figure 1.8
TOWER SECTION JOINTS - Anchorage section and plain joints

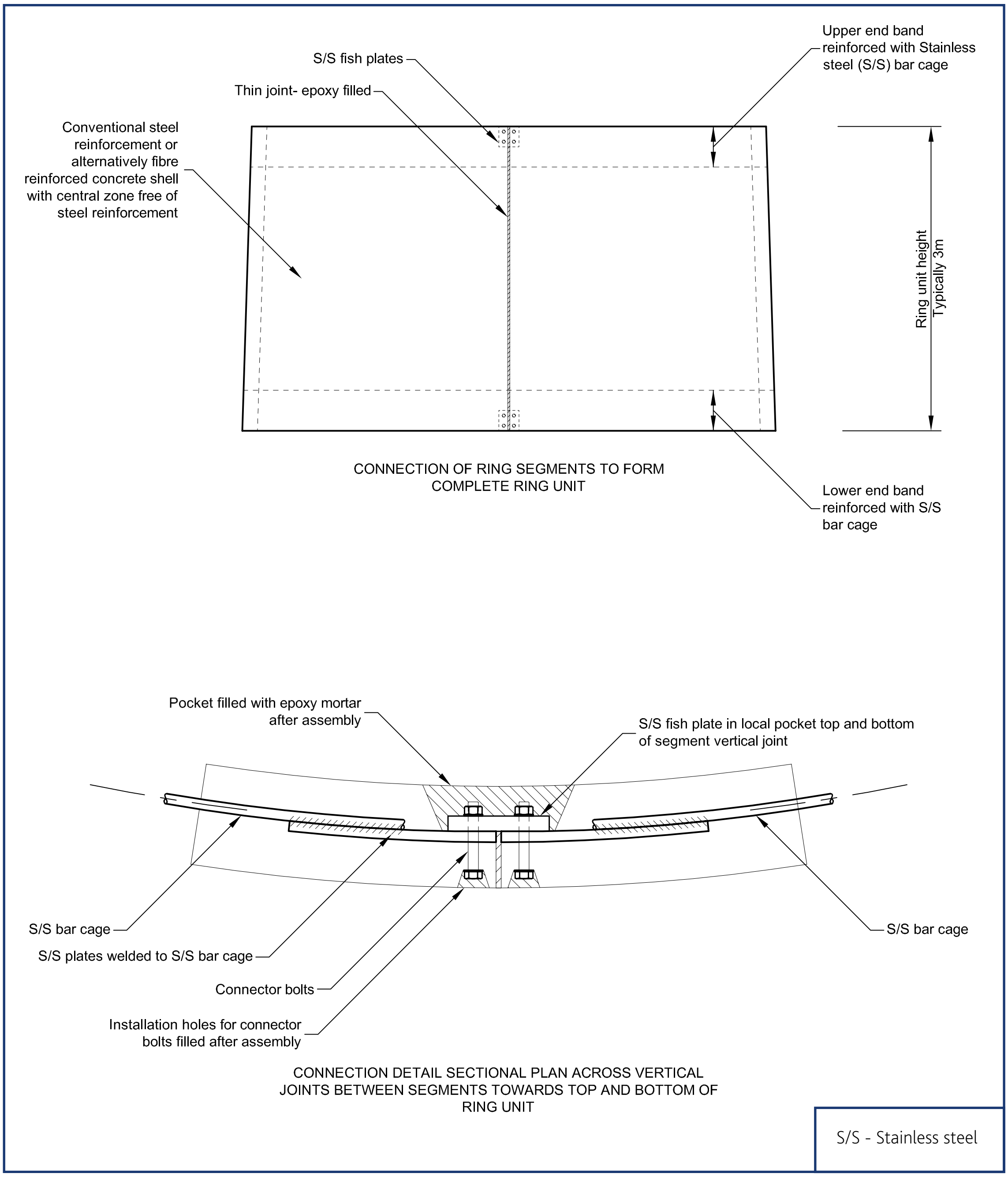


Figure 1.9
RING UNITS - Reinforcement and segment joints

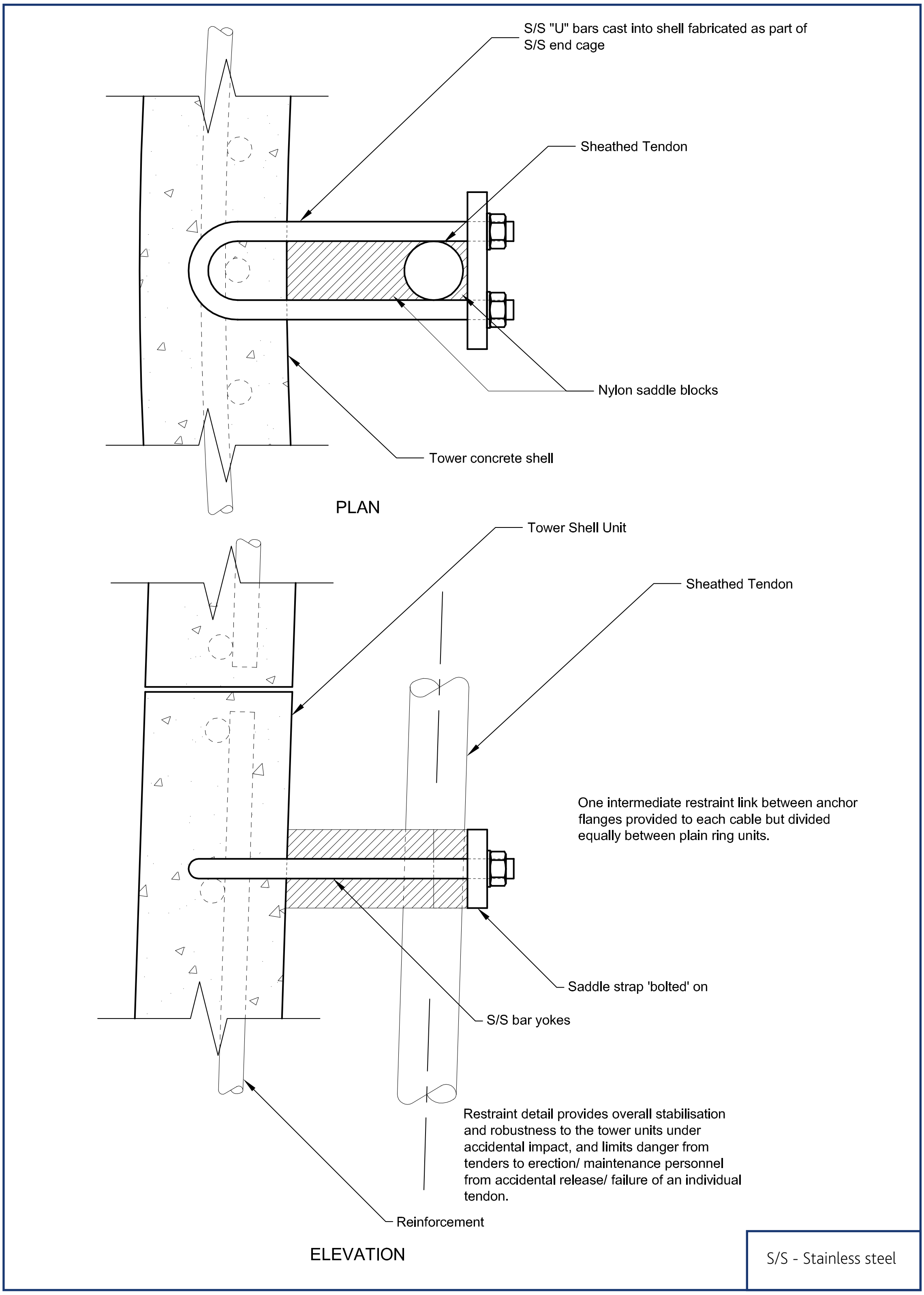


Figure 1.10
TENDON RESTRAINTS - Yokes to plain units

STUDY OF CONCRETE TOWERS FOR OFFSHORE WIND FARMS

This study was undertaken before the work on onshore wind farms. Some of the concepts touched on in this section have already been covered, particularly in relation to the design of a lighter weight pylon. Readers are advised to read both the onshore and offshore studies in order to get the fullest information from this report.

7 DESIGN PHILOSOPHY

7.1 Design drivers and approach

This section focuses on producing a design solution for offshore wind farms. The design scenarios considered make some allowance for the larger turbines and taller towers that are being developed to meet current trends. The aim is to illustrate that the conceptual concrete designs proposed are adaptable to both present and future industry needs.

Design drivers include:

- The size of turbines and rotors is increasing, demanding increased tower height, strength and stiffness.
- Offshore towers have to resist significant wave and wind loadings during storm conditions.
- Blade rotation leads to cyclic loading of both the blades themselves and the overall tower structure. Wave loading on the tower pylon also involves a high number of cycles with some regularity in frequency. Fatigue effects and tower resonance may both strongly affect design.
- Competitive and economic design requires simplicity of form and detail allied to a basically competitive technology.
- Aggressive marine environments require durable materials and details.
- Offshore operations are difficult, expensive and weather prone. Careful planning and design are required for ease and speed of erection and completion. Low maintenance and straightforward decommissioning are also important.
- Limited availability of offshore heavy lifting plant and the limited offshore construction season needs to be considered in design.
- For greatest economy, large scale production utilising production-engineered design is needed.

Life cycle issues are especially important for renewable energy systems. During initial design stages, general consideration has to be given to buildability - including fabrication, transportation and erection - based on current capabilities and practice. Reversal of the construction process to allow for eventual removal and disposal at the end of the tower's operational life must also be considered. These issues have been given considerable thought when developing the indicative designs.

7.2 Design scenarios and criteria

A generic scenario has been selected as a basis for preparation of trial designs. This is shown in Figure 2.1 and is intended to provide a reasonably representative but demanding test of both the present and future applicability of concrete designs for offshore applications.

Tower hub heights of both 80m and 100m above lowest astronomical tide (LAT) are considered in order to represent both common current practice and the tallest offshore wind towers likely to be constructed in the next 10 years.

A water depth of 25m (from LAT) is chosen. This is based on an assessment of all UK Round 2 wind farm sites[9]. Being at the deeper end of the range, 25m has been chosen to meet future as well as current demand.

The scenario is based on the trend of increasing generator sizes, which demand large rotor diameters and tower heights. It is anticipated that construction in water depths greater than those typical of the first generation offshore sites is likely to become necessary over the next decade as the availability of further shallow water sites diminishes, putting more importance on harvesting energy from sites that are currently less economic to exploit.

These further design criteria were applied for preliminary design purposes:

- A design life for the structure of 20 years and a return period for extreme storm events of 50 years were chosen, conforming to current practice.
- The critical design condition is the extreme storm event.
- Wave loading, wind loading and turbine loads are included. Wave induced/tidal current loads have not been considered since they are negligible compared to the extreme wave loading. A 3MW turbine is selected as the reference design case, as it is in the upper range of turbine capacity currently available. A commonly used turbine model has been considered, and the manufacturer's design loads have been used for the study.
- In accordance with the Intergovernmental Panel on Climate Change, expected sea level rise is taken as 0.005m per year, which is negligible for the purposes of this study. The effect of climate change on storms and on the recurrence of extreme values of wind speeds or wave height has not been considered for present design purposes.
- An allowance of two metres for seabed level changes (from scour, etc.) is included. Highest astronomical tide (HAT) is taken as 3.8m above mean water level, with an allowance for storm surge of 1m on top of HAT.
- An extreme wave height of 12m and wave period of 10 seconds are selected.

8 DESIGN

8.1 Tower configuration

For offshore situations, wind towers may be conveniently defined as having two distinct sections, namely:

- The pylon – from the nacelle to the level of the highest wave crest
- The foundation and substructure – everything below the pylon

Pylons experience substantial horizontal loads imposed from wind, through the operating turbine and directly on the tower stem. Additionally the substructure and foundations are subjected to larger horizontal forces from waves and possibly vessels. In comparison with structural forces imposed by turbine and pylon loads, the overturning moments imposed at foundation level by wave, and vessel loading in deep water sites (20-30m) are virtually an order of magnitude larger.

For deep water sites, it is likely that spread foundations – gravity or piled caisson, or tripod – rather than monopile, will best provide the required dynamic and structural characteristics. Caisson structures can be designed to be self-buoyant for tow-out and installation and, therefore, minimisation of self weight is not critical. Indeed, in such instances additional mass may be advantageous for stability. Nevertheless, it is important to consider limiting the mass of the material in the design for the sake of economy.

On the other hand, construction of pylons onshore with subsequent installation offshore might involve the use of cranage to lift large pylon segments. Minimisation of the pylon self weight will be important using such methods. The distribution of mass over the height of a pylon is also important as this is likely to contribute significantly to its dynamic response. This may need to be tuned in relation to the overall dynamic characteristics of the foundation/pylon system.

8.2 Pylon

A tapering hollow cylinder has been selected as the structural configuration of the main wind tower pylon. Recently constructed wind energy towers, both onshore and offshore, have predominantly used this simple and elegant form. It provides both an efficient structural member and an aesthetically pleasing appearance and, as such, is an appropriate starting point for this design exercise.

Variants on the tapering tube structure, or even perhaps some other form, might offer some potential improvements in efficiency, particularly in terms of specific strength or stiffness (i.e. strength or stiffness per unit weight of structure).

8.3 Foundation and substructure

Ground conditions are likely to vary considerably between different sites. Other site conditions may also vary, including water depths. In combination, the variations may demand a range of different foundation types and designs. To take some account of this, the study looked at two different solutions for concrete pylon foundations: a concrete gravity foundation and a steel monopile with a grouted concrete stem.

The concrete gravity foundation considered is a caisson filled with ballast or rock. The caisson would be circular or polygonal in plan. This is a tried and tested solution for underwater foundations which is potentially simple and for which there is a range of possible installation methods.

Where surface sea bed strata have sufficient strength, caissons can transfer loads through direct bearing. Alternatively, caissons could be piled for weaker underlying ground conditions, to increase the strength and stiffness of the foundation as necessary. The caisson would be set down on a prepared bed - usually comprising a levelled stone blanket. Typically the caisson may have a down-standing skirt or cutting edge around its perimeter. This would ensure a good horizontal key-in and provide some protection against bed scour and local soil weakness. An outer annulus of anti-scour rock may also be needed, depending upon the scour potential from waves, currents and bed mobility. It is remarked that scour protection is also an important and critical issue for monopile foundations.

An important aim is the achievement of maximum simplicity for gravity foundation designs and installation which can be applied to as wide a range of site conditions as possible. It will therefore be necessary to devise solutions which can, to a satisfactory extent, cope with a range of sea bed slopes, with poorer near-surface ground bearing strengths, and which allow the installation of scour blankets with minimum on-site preparation and relatively low sensitivity to wave and tidal conditions.

Whilst the initial focus for this study was the design of the tower superstructure, it is recognised that it is only one component of an overall structural system comprising:

- Bearing strata (ground/sea bed)
  Formation and protection at interface
- Foundation and tower stem (substructure)
  Interface formed by transition piece providing adjustment and connection
- Pylon (tower superstructure)
  Interface providing yaw ring/bearing, connection and adjustment
- Rotor and nacelle (carrying the hub and drive shaft, gearbox, electrical generator, ancillary equipment, controls etc.). The rotor and nacelle have been lumped together here for simplicity but for other purposes they might be better defined as a number of separate components

The conceptual and detailed designs of these interfaces/joints are fundamental to the success of the eventual system and need to be considered, to some extent, even at the initial feasibility stage. It is almost certain that the potential benefits from the use of concrete tower structures will be most fully realised in conjunction with the use of concrete offshore foundations and substructures.

8.4 Structural design methodology

Preliminary design has been based on a static analysis of the proposed towers under the chosen design loads. The dynamic response of the towers has been analysed to assess susceptibility to resonant excitation by either the turbine blades (cyclic variation in loading from blades passing the tower) or by consecutive waves hitting the tower. The tower's natural frequency has been calculated and compared with the range of the turbine blade passing frequency and the wave periods for the larger waves.

Natural frequency response of various types and dimensions of tower, both steel and concrete with different foundation conditions, has been estimated. The comparisons provide pointers to possible design issues for overall tower systems in deep waters, particularly where underlying soil properties do not provide sufficiently stiff support. It was identified that concrete tower pylon and foundation systems are likely to offer advantages over current steel designs in this regard, particularly as tower heights increase.

These comparisons are discussed further in Section 10.2.

8.5 Commentary on life cycle issues

The economics of renewable energy systems are dominated by initial capital, maintenance/operational and decommissioning costs, whilst 'fuel' is effectively free. This can lead to quite different cost/benefit balances and, therefore, different lifecycle strategies for design and operation from fossil or other fuel-driven systems.

In order to achieve the potential benefits of these different strategies, the design approaches and life cycle criteria for offshore wind machines need to be considered on their own merits. This is now becoming more widely understood. Although there is still some debate about the actual overall benefit in terms of CO_2 reduction, taking into account the variable nature of the output, there is a broad acceptance of the important environmental benefits of offshore wind generation. For the purposes of this study it is satisfactory to consider potential improvements on a comparative basis.

An interesting potential benefit from the use of a prestressed concrete design could be the increased life span of the tower structure. To fully realise this, the substructure would have to have similar durability. It is possible to think of re-fitting the turbine after its design life of 20 years with a next generation design over two, three or even four turbine life cycles, thereby avoiding the financial and environmental costs of reconstructing the tower and foundations. (Current thinking in Denmark in relation to steel towers is that they should be designed for a life of 50 years).

For the present exercise, the main interest lies in design concepts and approaches which can minimise production and installation costs. However, even at this early conceptual stage it is possible to envisage detailed design arrangements that could deliver life cycle improvements in this way. Other sections of this study describe possible production processes and design details which are potentially simple to fabricate and assemble, have good durability and reliability, and allow reversal of the erection process for eventual decommissioning.

8.6 Brief description of the outline designs

Figures 2.2-2.5 illustrate variants of the basic pylon design, allied to different substructure/foundation configurations. They provide for different site situations and a different balance between wall thickness and overall tube diameter, to achieve similar strength and stiffness performance.

It should be noted that no attempt has been made at this stage to identify an optimum configuration, which may in any case be strongly influenced by the characteristics and constraints of specific sites. Variables such as turbine loading and environmental conditions are likely to vary considerably depending on the project. Consequently the pylon solutions presented are considered to be a first design iteration.

8.6.1 Continuous taper design (Figure 2.2)

Within the lower pylon sections, the pylon diameter is kept down by the use of a wall thickness of 400mm plus. This allows a smooth tapering profile to be maintained through the upper wave zone and down the lower stem without attracting excessive wave forces.

For the larger water depths considered in this report, the strength and stiffness of the lower stem may be increased as necessary by flaring out the stem diameter or buttressing.

8.6.2 Necked stem design (Figure 2.3)

This alternative illustrates a different approach to maintaining the necessary pylon strength and stiffness. The maximum wall thickness in the pylon stem has been limited to 250mm and the pylon diameter increased accordingly. In order to avoid excessive wave loadings the tower has been 'necked' through the upper part of the wave-affected zone. The approach to maintaining sufficient strength and stiffness in the lower stem mentioned in Section 8.6.1 can also be applied here.

8.6.3 Concrete pylon with a concrete gravity foundation

The options in Figures 2.2 and 2.3 incorporate the use of a concrete gravity foundation. This type of foundation achieves stability by virtue of its mass and effective weight. Since the horizontal component of the principal operational and environmental loads may be applied with generally similar magnitude from any compass direction, a circular or polygonal plan shape is most efficient.

Simple gravity foundations transmit the vertical resultants of the applied loading (including self weight) by direct bearing on the sea bed foundation. The strength and stability of the upper levels of the sea bed strata have a particularly important effect on the behaviour of the foundation, although, as a rule of thumb, soil properties within a depth of 1.5 times the base diameter need to be considered.

Acceptable foundation diameters are considered on moderate strength soils. For situations where ground bearing strength is low, it is common practice to use piles to support the base and thereby limit base diameter. This option is notionally available offshore, but the installation of substantial numbers of integrally connected piles for each foundation would be impractical and uneconomic. Certain ground improvement techniques might be feasible and could widen the conditions for use of gravity foundations. This approach could be particularly valuable for dealing with marginal variations in ground conditions within particular wind farm sites.

Vertical load is necessary to provide acceptable sliding resistance and resultant bearing stresses, under the high horizontal and overturning forces.The additional weight of the concrete tower is advantageous in this respect. The foundation needs to be massive, particularly bearing in mind the buoyant uplift on submerged structures. It is economical to provide part of the foundation mass as ballast, in the form of dense rock or sand (if suitably contained).

Gravity foundations are conventional engineering solutions, both onshore and offshore, and have been used widely for towers and chimneys. The dominance of the horizontal loading component in wind energy structures (and the associated cyclic fluctuations) is towards the extreme end of the spectrum. Experience of long term foundation behaviour under these conditions is limited but this is true of any applicable foundation type.

The gravity foundation might take the form of a large cellular caisson with a base slab, either open topped or closed with a top slab. The open cells facilitate the placement of rock ballast onto the foundation after installation. A trial design for this type is shown in Figure 2.4. Design exercises have shown that this is probably not the most economical type for the larger diameter bases under consideration.

Alternatively the closed cells provide the possibility of buoyancy to assist flotation of the caisson for installation. A closed cellular foundation can also be permanently ballasted with sand, pumped into place, which generally would provide a cheaper solution than rock. Additionally, the lower tower stem may be buttressed against the caisson base to assist in the efficient transmission of loads from the tower into the foundation.

An efficient version of the closed cell concept can be achieved by flaring the tower stem into a large cone, with a base diameter approaching that of the required foundation base size. This gives a simple tower shell which is efficient in spreading the loads to the base slab, with a structurally efficient form for tower strength and stiffness. Additionally it can provide temporary buoyancy for the tow out condition, can be easily water ballasted as part of, or immediately following installation, and can be economically ballasted with sand for the permanent condition.

Base diameter for such conditions would be in the region of 30-40m, depending on the depth of water, site exposure and turbine rating. The presence of a foundation and tower structure projecting above the sea bed will affect the flow patterns and bed stresses in the vicinity of the tower, due to waves and tidal currents. Depending on the bed conditions, scour protection will be needed. This may be provided at various levels of protection, but particularly by means of a perimeter skirt to the caisson penetrating into the bed, in conjunction with an annular blanket. This blanket would need to be flexible and simply installed. It might simply comprise suitably graded stone or be a more connected arrangement of elements.

Gravity bases have not to date been used extensively for offshore wind towers. However concrete bases have been deployed successfully and economically on two or three substantial Danish wind farms (including Middelgrunden and Nysted). To date, there are no examples in UK offshore wind farms. A key feature of their potential lies in their simplicity. The opportunity exists for a relatively simple and quick procedure for installation, which could reduce time on site and weather risk. The achievement and exploitation of this potential simplicity is discussed in Section 8.8.

Realisation of the potential simplicity of offshore installation of the gravity foundations, coupled with exploitation of the associated installation benefit for the whole tower, could provide significant cost savings. The ability of gravity foundation designs to cope satisfactorily with the range of topography and bed conditions encountered in particular sites could have an important bearing on the number of sites for which this concept might be applied economically. There is a variety of design configurations and techniques which can enhance versatility. These questions can only be answered with real authority on the basis of further study. However there are good grounds to believe at this stage that there is sufficient potential benefit to justify those studies.

8.6.4 Concrete pylon on a steel monopile (Figure 2.5)

Concrete pylons could be founded on a steel monopile if desired and if foundation conditions allow. The design life of monopiles could be increased, if required, by additional corrosion protection; for instance by jacketing the most vulnerable upper section of the stem and/or with other measures such as cathodic protection. For larger wind machines and greater water depths, monopiles are reaching economic and performance limits, and alternatives such as piled tripod foundations or gravity foundations may be technically more suitable and cheaper.

Nevertheless, some large wind farm sites may encompass a wide range of foundation conditions with water depths varying between, say, 5m and 30m and bed soil profiles and characteristics also vary significantly. In these circumstances, it may be necessary to adopt more than one foundation type. Versatility and interchangeability of component designs could be valuable in such situations.

8.7 Pylon fabrication and erection

A significant part of the capital cost of wind towers will be determined by the offshore erection operation and the perceived level of risk associated with it. In order to achieve the lowest installed cost, it is important to recognise and allow for this and all other key fabrication and installation operations in the selection and development of conceptual designs. It is important to develop the engineering design to a high level of detail, with full regard to production engineering and the environmental conditions that might be experienced during erection. Careful attention must also be paid to risk minimisation through design of the structure and erection methodologies.

It can be concluded right from the start that it is not feasible to undertake extensive in-situ concrete construction work offshore on tower structures. The approach considered here relies on the use of precast concrete units and is broadly analogous to the current methods of offshore assembly of steel towers. The method proposed is not intended to be definitive, but provides a useful basis for reaching initial conclusions about feasibility and cost. This is described below and is illustrated in Figures 2.6 – 2.8.

8.7.1 Precast concrete method

The pylon is envisaged as being erected offshore from three or four large segments – depending on the available lifting capacity pre-selected for the project in hand. Other things being equal, it is preferable to use the lowest number of segments possible to achieve the required tower height in order to minimise the number of lifts. Transport to site may have a bearing on the decision.

Each segment would be pre-assembled from a number of shell units as illustrated in Figure 2.6.

A set of shells would be cast and then assembled into a segment, with prestressing strand threaded down the 'vertical' ducts cast into the wall of each shell. The strands would be prestressed and grouted to produce a monolithic segment ready for transport offshore. In prestressed concrete terminology, the segments would be post-tensioned bonded structures.

8.7.2 Slipforming the tower

Slipforming the tower is a continuous casting technique for building chimneys, silos and towers, which avoids the formation of joints and the use of heavy cranage. In the right conditions it can provide very economical structures, particularly where the special rig fabrication costs can be defrayed over a number of similar structures.

Slipforming offshore is not considered to be economically feasible. However, if transport and installation techniques can be devised that allow construction onshore, or inshore in sheltered waters, then it could provide an attractive option for tower (stem and pylon) construction. This is considered further in Section 8.8.

8.7.3 Prestressing techniques and details

The basic technique considered is the use of bonded (grouted) post-tensioned steel cables, housed within galvanized steel or plastic ducts. This requires sufficient wall thickness to accommodate and give proper concrete cover to the duct, which may limit the minimum wall thickness. This may be an unwelcome constraint in the middle and upper zones of the tower pylon. It also involves the grouting operation after prestressing, which, if it could be eliminated, would usefully reduce the work operations offshore.

Alternative approaches could involve the use of unbonded tendons, also in post-tensioning mode. These might be simply substituted for a bonded system and installed in the cast-in ducts with suitable protective systems; for instance, the normal sheathed strand for unbonded applications plus a duct fill of corrosion inhibiting grease/wax.

More radically they could be installed as external tendons on the inside face of the pylon shell. They would be restrained and linked back to the separate shells by suitable brackets at each shell joint to ensure composite action under abnormal horizontal or buckling loading. There would need to be a robust and reliable corrosion protection system in place, but given the simplicity of the system this should be practicable, particularly for the tower pylon.

It would be possible to provide a further line of defence by introducing some active control of the tower's internal relative humidity. This might involve (and also benefit) climate control of the nacelle. Such approaches are being considered for steel towers and are thought to be relatively simple and cost-effective.

Potential advantages of external cable systems would be great simplicity, economy and speed of installation, coupled with inspectability, replaceability and ease of dismantling pylons at the end of their life cycle.

8.7.4 Ancillary details

There would of course be a number of detailed finishing operations; for example the installation of a light steel stair/lift tower. These would include internal landing platforms to assist in the offshore operations during placement and joining of segments. This light internal structure would also serve to provide access for subsequent inspection and maintenance of the turbine assembly and of course, the tower itself. Similarly, certain other fixtures could be made, such as the J–tube/electrical cable conductor tubes (whether of external or internal arrangement), the boat landing platform for maintenance and general access.

8.7.5 Formwork for precast units

Formwork to produce the set of tapering units needed for each tower represents a significant setup cost. Given sufficient initial investment in formwork and other production equipment and facilities, repeatable accurate surfaces and dimensions can be achieved without a major cost premium. This investment needs to be defrayed for a significant volume of business (i.e. sufficient numbers of units to be produced on an essentially continuing basis).

For instance, given 50-100 reuses for formwork, fabricating high accuracy steel formwork to produce coned concrete shells/segments should be economic. (Conical steel formers may be produced by plate shaping and roller techniques similar to those used for forming the structural shells of steel towers.) Such volumes are available within a single project of the size of the large Round 2 wind farms.

8.7.6 Joint details

The aim is to keep joint details as simple as possible, both in form and in assembly. Joints between shell units within segments are illustrated in Figure 2.9. These joints are based upon the use of a thin epoxy resin filler, with material being placed before the positioning of the upper shell. It is assumed for the moment that vertical assembly would be most suitable. Accurate alignment and flatness of shell unit joint surfaces will be required. It is envisaged that assembly will take place under controlled and efficient conditions with an assembly frame providing access, weather protection and jigging. The segments will be prestressed in this assembly position and the steel strand grouted into preformed ducts, with post-tensioning in the normal way.

Joints between segments would need to be made in the more exposed and difficult circumstances of the offshore site – unless the erection/installation approach discussed in Section 8.8.3 is adopted. It is important that the segment being erected can quickly be lowered and accurately placed into position and secured. This may involve detachable temporary guides or fixings. The joint detail shown in Figure 2.10 is intended to allow for this. It may be possible to use a number of long prestressing bolts to provide initial fixing, with a nominal prestress prior to sealing and grouting the 'thicker' in-situ epoxy joint.

Accurate vertical alignment of the upper unit could possibly be achieved by placement onto a limited number (say three) of slightly raised and accurately pre-set bearing pads. These might be stainless steel plates or epoxy concrete. The joint would be edge sealed and injected with epoxy grout. It would be necessary to design this operation to be undertaken quickly and simply and for it to cure sufficiently rapidly to allow early stressing.

During shell and base segment assembly, a proprietary gasket or connector might be used to achieve alignment of the duct tubes and sealing against epoxy grout intrusion. The prestressed bolts could be left unbonded, but would be end-capped and the ducts filled with protective grease. This could provide not only good protection, but would allow sample bolts to be inspected during their lifetime, allow any replacement if necessary and facilitate eventual decommissioning.

There are, of course, other ways that joints could be made and a more thorough study of joint design would be an important part of the next stage of work for a precast method.

8.8 Construction and installation of the foundation and tower

8.8.1 Current installation methods for steel towers

For offshore wind farms the predominant structural configuration at present is steel tower/steel monopile foundation. The installation of offshore wind towers currently involves at least two distinct major operations at the offshore site, namely:

- Foundation - substructure installation
- Erection of the tower – pylon, nacelle and rotor

Monopiles comprise heavy, large diameter, steel tubes driven to considerable depth (typically 25-30 metres plus) where conditions allow. Driving through glacial deposits (boulder clay, moraines etc) or weathered rock can be problematic and uneconomic.

Where there is underlying rock at shallow depths, the pile may need to be housed into sockets cut into the rock and grouted. Commonly, an adjustment piece comprising a substantial length of oversize tube is fitted over the driven pile. This tube provides the arrangement for correcting any deviation from verticality in the monopile and has a connection flange at its top end for the connection of the tower above. After setting to the required vertical alignment, the annular space between the adjustment tube and the pile is cement grouted to provide the required structural connection.

Purpose-designed installation plant has been commissioned in the past two years or so, including various jack-up platforms for monopile handling and driving, and jack-up ships for transporting and erecting pre-assembled pylons, nacelles and rotors in one or several lifts according to size. Tower installation vessels are capable of transporting several (three or four) towers on one trip. In suitable conditions, this plant can complete these operations efficiently within quite short periods on-station.

It should be possible to use the current specialised plant for tower configurations using steel monopiles and concrete pylons. Even so, erection cycles for a concrete pylon may be a little more extended, due to the greater number of lifts per pylon dictated by the maximum crane lift capacity. With careful planning and optimisation of the concrete pylon weight, this may not involve any serious increase in the on-site time and weather risk.

8.8.2 Installation of gravity foundations

There are many situations where gravity foundations might be used as an alternative to monopiles. The use of concrete gravity foundations for offshore wind energy converters has so far been limited to two or three projects in the Baltic Sea with relatively shallow water depths and where winter sea icing has been an issue. Typically installation has been undertaken efficiently using adapted flat top barges carrying the foundations as deck cargo, along with separate floating cranage for placement. Towers and turbines have been erected later as a separate operation as is normal for monopile foundations.

Gravity foundations (concrete caissons) for deeper water would necessarily have to be quite large, typically with a diameter of 35-40m (being circular, octagonal or hexagonal in plan). The installation methods adopted to date would involve the use of very heavy offshore cranage and involve a prohibitive cost.

It is understood that the use of concrete gravity foundations for the Baltic Sea projects has resulted in useful cost savings. For similar circumstances, it is possible that the techniques used to date could be further improved to good effect. However, they are unlikely to be sufficiently economical for the greater proportion of future offshore wind sites where deeper water and more severe wave conditions will be encountered, particularly in view of the large foundation sizes and weights likely to be associated with these conditions.

In order to achieve the substantial cost and risk reductions that are needed to attract investment for the offshore wind programme, a radically different approach that addresses the major cost and risk centres must be found.

There is currently no purpose-designed plant to handle such units although, with some ingenuity, it should be possible to create facilities and plant at a viable cost.

Caisson foundations in nearshore and offshore situations have a long history in civil engineering practice and also in the offshore oil industry over the past few decades.

8.8.3 Alternative proposals for caisson and tower construction and installation

As mentioned in Section 8.7.2 it should be possible in principle to achieve a very efficient process with installation work offshore limited essentially to the placement of a completed tower structure in one operation, thereby avoiding the potential weather risk from extended erection activities on the tower superstructure. This would involve assembly and erection of the foundation and tower structure inshore, followed by a tow-out of the whole unit as a single entity.

Siting a shore-based waterside production facility where there is access to reasonably sheltered deep water, such as an estuary-located port or shipyard would allow the following method to be used:

1) Construct the caisson and tower stem on a waterside construction berth (with suitable launch arrangements) or in a wide dry dock.

2) Launch or float-out caisson with suitable temporary flotation attached for stability.

3) Sink onto prepared harbour bed pad in sheltered location (water depth say minimum eight to ten metres).

The construction of the tower onshore or in sheltered inshore conditions opens up the possibility of using two quite different methods of tower construction as alternatives: a) precast concrete units/ segments, b) slipforming (see Sections 8.7.1 and 8.7.2).

4a) Precast methods. Construction pads might be arranged alongside a quay with suitable water depth (say 8-10m including neap tidal range) or either side of a finger jetty projecting from the shoreline. The quay/jetty would carry a travelling gantry crane of sufficient capacity to lift at least the largest single precast units, (say 50 tonnes), or ideally the fully fitted or substantially fitted out nacelle (100-250 tonnes plus depending on turbine type/rating). The capacity of the gantry crane could be substantially reduced to, say, 60 tonnes if the requirement was limited to lifting individual precast concrete tower units, leaving the turbine (rotor – generator etc.) to be lifted by crane barge, either inshore or offshore, depending on the preferred methodology. Similarly, a smaller crane barge for lifting the precast units could provide a more economic lifting facility for situations where the number of tower units to be produced was insufficient to support the investment required for a more efficient facility.

4b) Slipforming method. Close proximity of construction pads to a quay or jetty would be less important as the tower slipform operator in the 'inshore' pads could be supported by a simple floating barge or jack-up platform, to provide accommodation for personnel, deck storage for materials delivered by barge and space for a small concrete batching/mixing plant. The slipform rig would probably be attached to the base caisson onshore and sufficient height of tower stem/pylon would be completed before launch and movement to the pad. Once the caisson was settled down on the (temporary) construction pad, slipforming would resume until full completion of the tower.

The next stages of lifting the caisson/tower structure off the pad and transporting to site would proceed similarly for each alternative.

5) A catamaran barge with an overall width of about 40m would be moved to straddle the tower and lift the caisson and the whole tower unit off the pad for transport offshore. The caisson/lower tower would be connected to the barge using lifting tendons well distributed around the structure. Lifting might be achieved by, say, strand jacking or might be achieved or assisted by de-ballasting the barge or the effect of the rising tide.

6) The design of the shape and sizing of the 'catamaran' barge is itself a matter of detailed engineering and naval architecture study. The outcome would depend upon the range of tower unit sizes and site water depths to be catered for.

The tower caisson configuration could also affect the lifting/carrying capacity needed, as some designs could provide substantial buoyancy assistance, reducing the required capacity from, say, 4000-5000 tonnes to 2000-3000 tonnes. For instance, sufficient capacity for the latter case could be provided by twin hulls, each of 60m x 10m x 5m depth, operating at a maximum draft of, say, 3-3.5m. This spacing of the hulls would be determined by the tower stem/upper caisson shape and dimension. Allowing for buttressing or significant flare on the tower stem, a separation of some 20 plus metres may be required.

The tower would be lifted off the pad and moved to deeper water if necessary, where the draft to the underside of the caisson unit could be adjusted for the tow offshore. A draft of 8-10m is expected to give good stability for open sea towing.

7) The whole unit would be towed by tugs to site to arrive during a predicted weather window, suitable for lowering or sinking the tower onto the bed (prepared as may be necessary).

8) Rock barges or a sand dredger would rendezvous at the site to ballast down the caisson once the catamaran barge had cleared the site, to give full storm stability to the tower.

This method could be used for constructing and placing a concrete caisson unit with stem, ready for in-situ erection of a steel or concrete tower, if circumstances favour such an approach. The facility for constructing and launching the substructure unit (caisson and stem) alone would not need the cranage facilities for building the pylon. Figure 2.11 illustrates this latter situation, whilst Figure 2.12 shows the in-shore construction and tow out of the whole tower.

The methods illustrated provide different methodologies for offshore installation of a pre-assembled foundation/ tower at varying degrees of completion. They include the possibility of achieving the ultimate level of prefabrication, by the inshore completion of a fully fitted out and equipped caisson foundation/tower/turbine unit pre-tested and commissioned before tow out, requiring only limited finishing operations offshore, including final ballasting, scour protection, and hauling in and connection of the electrical cabling.

This may seem too ambitious at this stage, but it nevertheless appears to provide a pathway to maximum economy for offshore wind farms, based upon horizontal axis wind turbines as conceived at present.

8.9 Versatility of the concrete tower concept

The design concepts may be applied in a wider way than simply presenting concrete as the alternative solution for the entire wind tower. Concrete foundation elements and tower elements may be used separately in combination with steel structures in a variety of configurations. Options include:

- Concrete pylon with a concrete foundation.
- Concrete pylon grouted onto a steel monopile.
- Steel pylon bolted to a concrete foundation.
- Concrete/ steel hybrid tower onto any of the above foundation types.

Figure 2.13 illustrates the versatility of the concept.

Up to now, steel monopile foundations and towers have dominated the offshore scene. The projected major increase in the installed capacity of offshore wind energy generation over the next decade or two provides an opportunity for a wider range of structural types and designs to be used to achieve optimum economy.

The ability to use hybrid material combinations for foundation/tower structures, and even within the tower pylon itself, provides significant flexibility for design optimisation for particular projects. It allows better design tuning for a wide spectrum of static and dynamic loading, for site circumstances (water depth; wave climate; bed and foundation conditions), logistics, and the prevailing economics of construction, plant availability and material supply.

Certain hybrid combinations, besides having their own intrinsic merits for particular circumstances, may also be used to provide a series of development stages prior to the deployment of the fully assembled and outfitted concrete foundation/tower units proposed in Section 8.8.

8.10 Standardisation of pylon geometry

The achievement of a high level of re-use of special production equipment is likely to be a key part of cost competitiveness and reduction. In the case of the tower pylons constructed using the precast method, this would concern the formwork/ moulding system, for instance, which would necessarily be purpose-made and represent a significant initial investment. At the most basic level the moulds would have a fixed geometry with little or no adjustment. These could therefore only produce a particular tower profile (maximum/minimum outer diameter and taper).

The production of a tapering tower barrel (for chimneys and other towers) is routine in slipforming practice, with well-proven arrangements for continuous hydraulically-driven adjustment of the diameter of circular section forms. Similarly adjustable forms are possible for the production of precast units. The balance between the complexity, and therefore the cost, of adjustment mechanisms and the benefits is a specialist technical matter. However the economics will need to be predicted from the costs of acquiring and operating the formwork system, its actual output as judged from its potential output and likely utilisation.

It can be reasonably supposed that some effort to define a standard profile or standard range of profiles, taking account of these considerations, is likely to be beneficial to promoting the wider use of concrete pylons. Given that considerable finer tuning of the design can be achieved relatively easily by adjustment of concrete strength, prestress level, reinforcement and wall thickness, it may be possible to define a limited set of standard profiles that are suitable for a wide range of tower heights and duties.

8.11 Prototype and demonstration designs

A number of onshore concrete tower pylons of considerable size (height 80-120m) have been constructed in Germany in the past five years, using both precast and in-situ slipform techniques. Concrete gravity foundations have been used on at least two or three Danish wind farm projects in the Baltic Sea.

No concrete wind tower pylons have been used offshore anywhere in the world as far as is known. However it is remarked that a substantial number of very large concrete production platform structures have been deployed in the North Sea for the last two or three decades. These include major slipformed concrete towers supporting the deck.

It will be almost certainly necessary to produce a number of prototype concrete structures at full scale or near full scale for development purposes and for credibility and confidence. This is likely to be the case for the main elements - foundations and tower pylons - both separately and in combination.

For concrete tower pylons the easiest prototype development route would be their use in onshore wind farms. This should reduce development costs significantly by offsetting the prototype costs against the market price of the 'conventional' solution.

If a precast concrete solution were to be adopted, useful guidance on various practical issues might be gained from experience in Germany, from discussions with specialist formwork manufacturers and from specialist fabricators producing conical and tapered steel towers. The alternative technique of in-situ slipform construction could probably be more easily adopted for single tower or a limited production run. The unit costs could be significantly reduced for the latter case.

9 QUANTITIES AND COSTS

The object of this study is to draw initial conclusions about the feasibility and cost of concrete towers for offshore wind energy converters. Relative costings are necessarily based upon educated judgement of the allowances to be made for both 'positive' and 'negative' uncertainties. Quantified estimates have been made upon simple model designs for the tower and foundations. Some allowance has been made for possible improvement in the design. However, realising the benefits of scale, possible improvements from quantity production and design refinement are set against the uncertainties connected with such things as the cost of the particular offshore operations required.

At this early stage, absolute cost evaluations are likely to be less reliable than comparative costings between 'concrete' and 'steel' solutions. Even more so, since there remains considerable difficulty in obtaining actual cost information on components of wind energy systems, despite the now substantial experience of constructing offshore wind farms, as this is a closely guarded commercial asset for the main providers.

Various alternative 'model' designs have been selected for the mast. Trial designs for a concrete caisson to act as a gravity base foundation have also been undertaken as a base point for costing assessments. Our costings of the pylon designs indicate that the installed cost of concrete pylons could be competitive with steel pylons, provided that the initial investment costs for formwork and unit production were spread over a sufficient production run of, say, 50 units.

Similarly we conclude that concrete caisson foundation units, apart from special circumstances in shallow waters such as those in the Baltic Sea, could be competitive with alternative monopile, or more particularly, tripod foundations for deeper water sites to about 20-30m and for larger rated turbines, say 3.5MW plus. As for towers, the efficient production of the larger caisson units associated with these conditions is likely to require the deployment of purpose-designed launching and transport arrangements. Production runs of 100 to 200 units would probably be required to defray the capital costs sufficiently.

The possibility exists for deeper water sites to construct a concrete gravity foundation and concrete pylon as an integrated unit in a sheltered inshore location, fit it out completely and then tow it offshore using a catamaran barge 'straddle' carrier, settling it onto a prepared bed in a single operation. This integrated approach to installation could offer a route to substantial cost reduction if applied on a sufficiently large scale as mentioned above.

Our assessments suggest that savings of 20-30% could be available in both the build and installation cost over current methods. The foundation/ tower build costs and the associated offshore installation, including the installation of the turbine, represent about 40% of the total installed capital cost for a wind farm. The table illustrates the range of saving to the overall capital costs that is potentially available from the 'integrated' approach using concrete foundations and towers as described in this study.

Item	% item cost reduction	% item cost to total capital expenditure	% capital expenditure reduction
Construction cost	25 - 50	20	5 - 10
Installation cost	25	20	5
Potential reduction			10 - 15

If this assessed potential can be shown to be achievable for a sufficient number of wind farm sites by more detailed feasibility studies, then this method and similar approaches could make a very substantial contribution to the cost and risk reduction needed to move forward new investment in UK offshore wind farms.

10 COMPARISON OF STEEL AND CONCRETE TOWERS

This section provides an overview and comparison of the features and relative advantages of steel and concrete towers. Although this section was prepared specifically for offshore towers many of the points are relevant to onshore towers.

10.1 Comparison of the dynamics of steel and concrete towers

Existing steel monopile wind towers have natural frequencies falling comfortably in between the blade passing frequency and the wave excitation zone. The newer generation of taller steel towers will have longer natural periods of oscillation than currently installed designs which move their natural period into the wave excitation zone (assumed to be six seconds and above). Larger and heavier turbines will also give the towers longer natural periods, exacerbating the problem. A chart comparing the natural frequencies of various steel and concrete towers has been produced (Figure 2.14).

With different foundation conditions, concrete towers on steel monopile foundations would potentially have a similar problem to steel monopile towers. Concrete can be more easily and cost-effectively adapted to larger diameters to produce stiffer towers to combat this potential problem.

The stiffness of the overall tower structure depends upon the stiffness of the pylon, tower stem and foundation. Monopile foundations have relatively limited structural dimensions and a restricted interface with the supporting soil. This considerably limits the stiffness achievable, particularly on poorer soils.

Gravity foundations involve more of the surrounding soil in resisting loading than the monopiles. They therefore reduce the natural period of the concrete tower system compared to a similar tower with a monopile foundation. This is advantageous for the taller wind towers and provides more scope for tuning the design to bring the natural period back in between the two excitation zones.

Concrete used for either the pylon, foundation or both can therefore offer advantages when designing for the dynamic performance of the tower. Concrete also has higher material damping properties than steel. Prestressed concrete has a high fatigue resistance, providing more tolerance and therefore less risk from dynamic problems.

10.2 Comparison Table of Offshore Tower Structures' Characteristics

STEEL TOWER	CONCRETE TOWER
Onshore fabrication/construction	
• Well-established technology. • Fabricate sections in factory under controlled conditions. • Prefabricate in large sections, with welding at shore location. • Larger diameters (5m+) and higher wall thicknesses (60mm+) required for deeper water and larger turbines become relatively more expensive to fabricate.	• Construct at factory facility at suitable coastal site. • Concrete can achieve larger diameters without disproportionate increase in cost. • Concrete can accommodate detailed section changes relatively easily. • Significant initial investment in formwork. • Formwork can be reused over long production runs giving lower unit costs. **Pylon stages** • Precast method (construction offshore or inshore). Precast ring units assembled into larger segments for installation offshore. • Slipform method (inshore only). Form tower on gravity caisson for tow out and installation offshore. **Gravity base stages** • Construct in dry dock or on/at top of slipway by shore. • Flood dry dock or launch from slipway/ shiplift. • Complete top sections with base on seabed at quayside.

Comparison Table of Offshore Tower Structures' Characteristics (continued ...)

STEEL TOWER	CONCRETE TOWER
Transportation	
• Larger diameters (4.3m+) and weight of sections become harder to transport by road from fabrication shop to coastal yard. • Relatively lightweight overall (order of 500 tonnes overall for a 90m pylon). • Can be transported to site on barge, several at a time. • Can be transported to site by floating out with sealed ends.	**Pylon stages (if separately transported to offshore site)** • Lift onto barge or launch down slipway. • Transport horizontally or possibly vertically to site on barge or float out in segments with ends sealed. **Gravity base stages** • Lift with special catamaran barge and transport (part submerged) to site. **Pylon erected inshore** • Built onto gravity base. • Whole assembly transported and placed by special barge.
Installation	
Equipment • Install using a large jack-up barge, shear-leg crane barge or specialised jackup vessel (e.g. Mayflower). • 300 tonnes or more can be lifted at present. **Stages for steel monopile installation** • Install monopile and scour protection (if needed). • Transition piece grouted to monopile head to form vertical connection. **Stages for pylon installation** • Install first pylon section, bolted to adjustment piece. • Install second pylon section. • Install nacelle and rotor blades. • Install electrical cables test and commission. **Features** • Relatively light weight per unit length. • May require fewer lifts (two pylon sections). • May therefore require shorter weather windows for installation. • Tried and tested technology.	**Equipment** • Install using a large jack-up barge, shear-leg crane barge or specialised jackup vessel (e.g. Mayflower). • 300 tonnes or more can be lifted at present. **Stages for steel monopile foundation installation** • Install monopile and scour protection. • Junction piece grouted to monopile head to form vertical connection. **Stages for gravity base installation** • Prepare seabed • Lower or sink from submerged position held by catamaran barge. • Place loose rock ballast into caisson cells for long term storm stability. • Place scour protection. **Stages for pylon installation (if undertaken offshore)** • Install first pylon segment. • Install second pylon segment. • Install third pylon segment. • Install nacelle and rotor blades. • Prestress, test and commission. **Features** • Greater weight or unit length than steel pylon requires more segments and correspondingly more lifts for complete erection. • Only three segments are needed when using large lifts of 300 tonnes. • With careful connection detailing and erection planning increased disruption risk may not be significant. **Alternative method** • Inshore construction of foundation tower unit inshore. • Complete unit tow-out option simplifies offshore operations. • Sinking and placement of whole unit essentially similar to gravity base operation above.

Comparison Table of Offshore Tower Structures' Characteristics (continued ...)

STEEL TOWER	CONCRETE TOWER
Performance	
• Static loading: plenty of reserve capacity in section. • Dynamic loading and response: the natural periods of present designs (circa six seconds) lie between the blade passing period (one to two seconds) and the periods of extreme wave conditions (six seconds plus). • Taller turbines need larger pylon and foundation diameters, providing sufficient stiffness to control the dynamic response (i.e. to keep the natural base period of the pylon-foundation system below the periods of extreme wave conditions). • Larger diameters will require larger wall thickness, making steel harder to procure and fabricate than hitherto, as well as heavier. • Larger diameter elements will be more difficult to install. • Connections (flanges etc.) – lives are limited by fatigue.	• If supported on a monopile foundation, require large diameter for concrete solution to avoid wave period resonance. • A spread foundation (gravity or piled) in contrast has potential to give good dynamic response for a large height tower. • Low maintenance. • Prestress can improve durability and provide a good fatigue performance. • Concrete has higher material damping than steel: potentially more tolerant of occasional resonance and avoidance of 'ringing' due to impact by breaking waves. • Significant potential for further value engineering: use of higher strength concrete; use of fibres to reduce vertical rebar and improve durability; use of carbon fibre reinforced polymer (CFRP) prestressed concrete.
Maintenance	
• Corrosion protection: high specification paint systems have about 15 to 20 years to first maintenance. Repainting may therefore be necessary if a design life of more than 20 years is required, particularly in the intertidal and splash zones. Cathodic protection below mean sea level will also require periodic maintenance. • Maintenance is difficult in isolated offshore structures. • Bolted flanges particularly at lower levels are vulnerable to corrosion.	• Highly durable if good quality construction. • Very little maintenance for structure.
Upgrade	
• Offshore wind energy converters have a typical design life of about 20 years. It may be possible to extend their life if required, but control of corrosion and of fatigue damage at connections could present significant problems. If the foundations and pylon are still serviceable, then the turbine could be replaced with a newer model.	• Concrete can be very durable (design life 50 to 100 years), thus the reuse of tower by upgrading the turbine at 20 years is highly possible. At this stage, the foundation and pylon, which may originally comprise some 40% of the overall cost of a new development, is almost "free". • A prestressed concrete section could be strengthened relatively easily for potentially higher loads, from newer larger turbines by providing some margin in strength of concrete selected, and by subsequently increasing the vertical prestress force by installing 'external' unbonded tendons coupled to the inner face of the pylon walls.
Decommissioning	
• Large diameter monopiles cannot be extracted at present; therefore they will have to be cut off at or below seabed level. • Steel of tower can be brought onshore and recycled.	**Offshore assembled towers** • Pylon unbolted in sections and returned to shore for disposal or recycling. • Gravity foundation could be deballasted and floated back to shore for decommissioning. • Large diameter monopiles if used cannot be extracted at present; therefore they will have to be cut off at or below seabed level. **Inshore constructed tower/ foundation units** • The whole installation can be reversed in effect, utilising equipment similar to that for original installation.

11 OFFSHORE STUDY CONCLUSIONS

The main conclusions are:

- Offshore wind farms are generally becoming larger, with a move to more units in deeper water, and deploying larger turbines. This trend points to the need for taller and stronger towers.
- An outline design of a prestressed concrete tower has been prepared for use offshore with the larger turbines likely to be used in the Round 2 offshore programme. The design is considered to be feasible and economic, although it will require further development to achieve optimal competitiveness.
- Concrete solutions offer advantages and disadvantages for the various scenarios for current offshore wind energy converters. The balance of advantage for taller offshore wind energy converter towers is shifting towards concrete design solutions.
- Concrete gravity foundations are competitive with steel alternatives (tripod system) in deeper water (particularly in the range of 15-30 metres), although the actual advantage would depend on the specific site conditions.

  An integrated design of concrete caisson and tower could be assembled in sheltered inshore waters, towed out as a completed unit and sunk at its offshore location. This approach could reduce substantially the number of critical offshore operations and thereby the associated weather risk. It could provide a radical alternative to current offshore wind farm construction methods and potentially could provide significantly lower out-turn costs than other existing deeper-water solutions such as the steel tripod foundation.
- Concrete design solutions have a potential for longer reliable operational lives (50-100 years) allowing reuse of the tower and foundation structure for later re-fits of the turbine and mechanical/electrical plant. They could thereby give significantly improved longer term financial returns and substantial improvements in sustainability.
- There is a very large programme of offshore wind farm development planned for UK and European coastal waters over the next five years. There is likely to be a continuing programme beyond that, with an increasing proportion of sites to be found in deeper waters further offshore. All this implies the need for a significantly increased industrial capacity to achieve it. The introduction of concrete designs as an alternative to steel could offer the important benefit of mobilising a wider sector of the industrial and technological resources of the civil engineering industry. This would tend to maintain or extend competition and provide more options for accommodating local economic and employment circumstances.

12 FIGURES - OFFSHORE WIND TOWERS

Figure No.

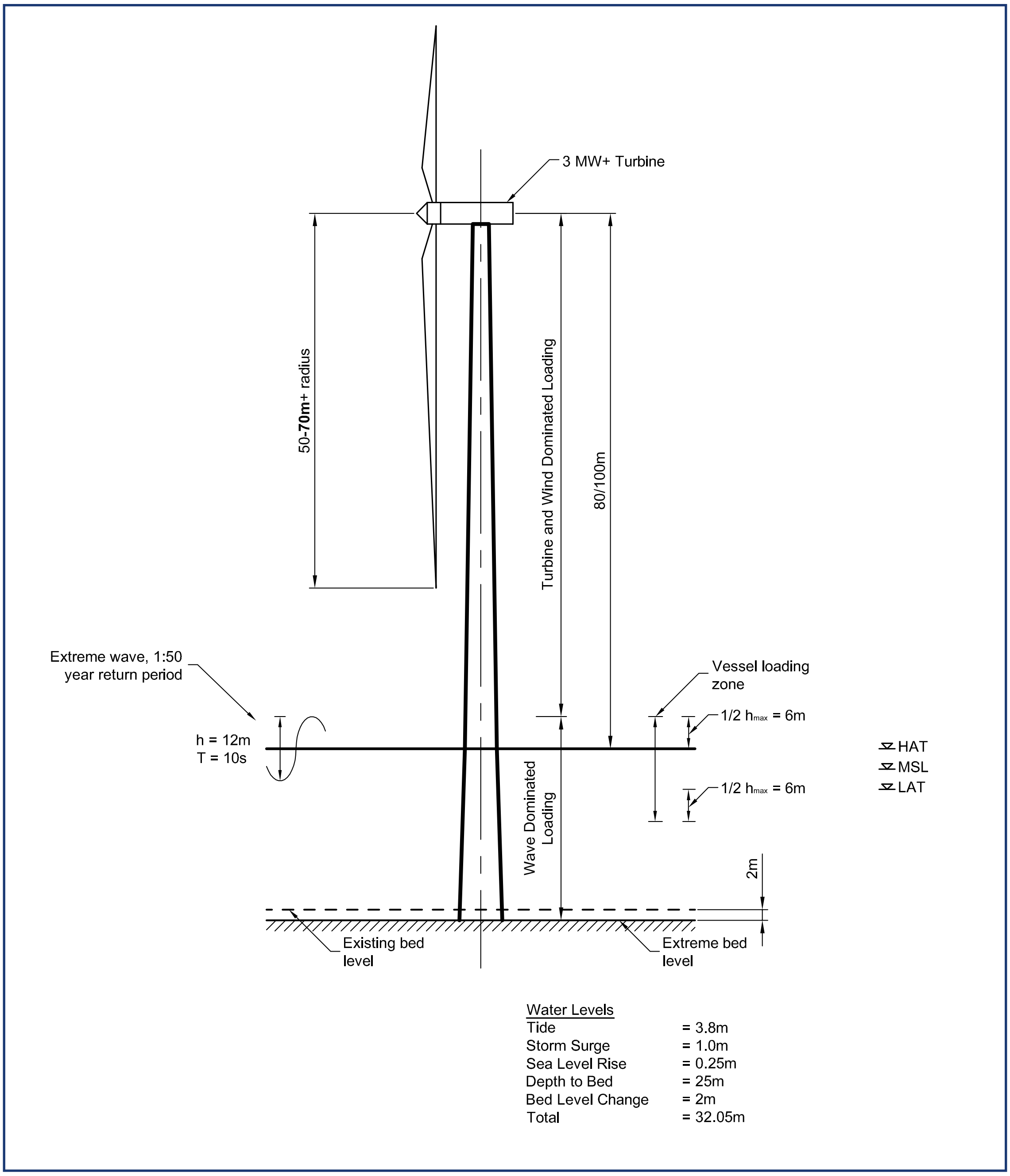


Figure 2.1
GENERAL STUDY SCENARIO

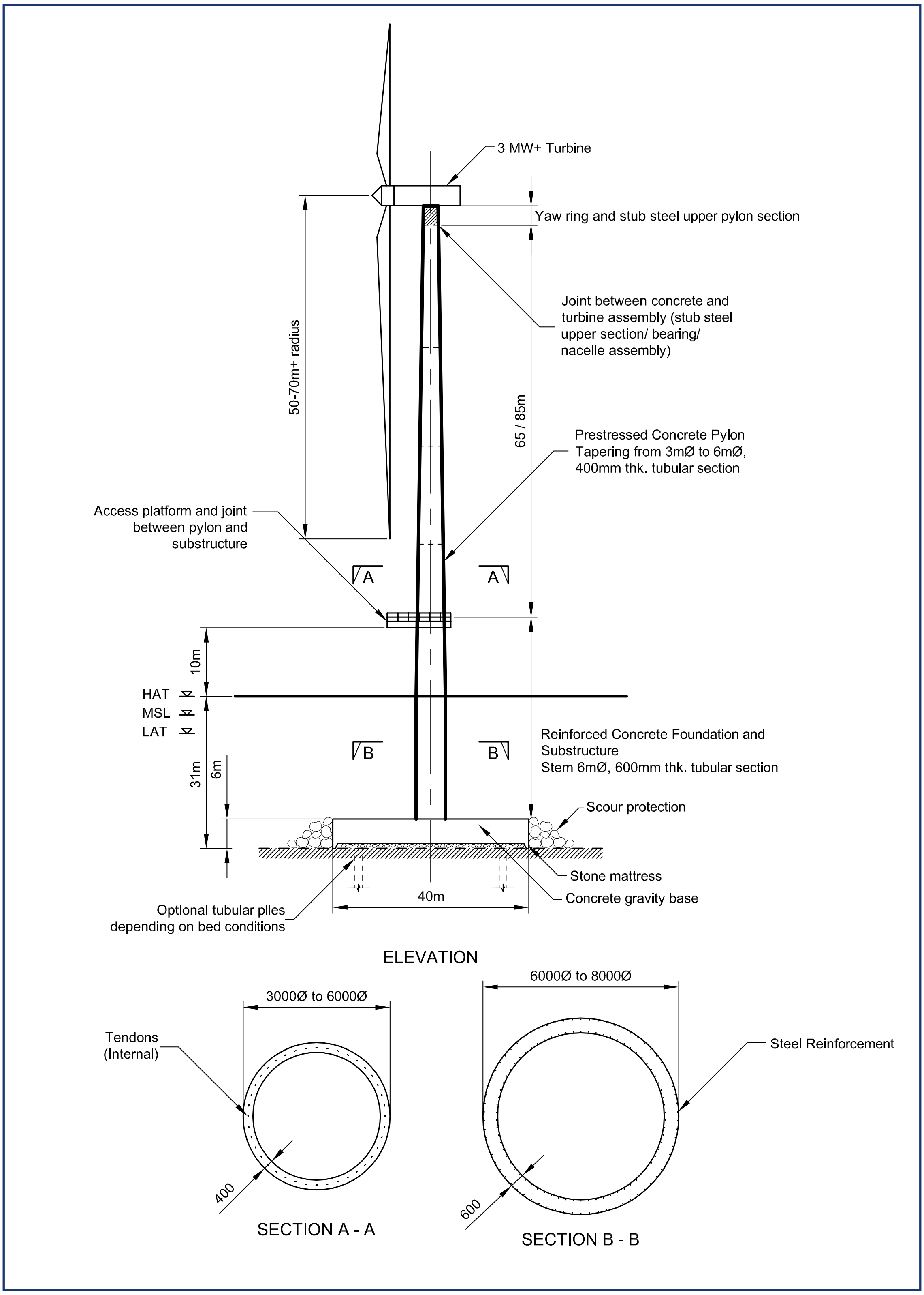


Figure 2.2
CONCRETE TOWER - Continuous taper design

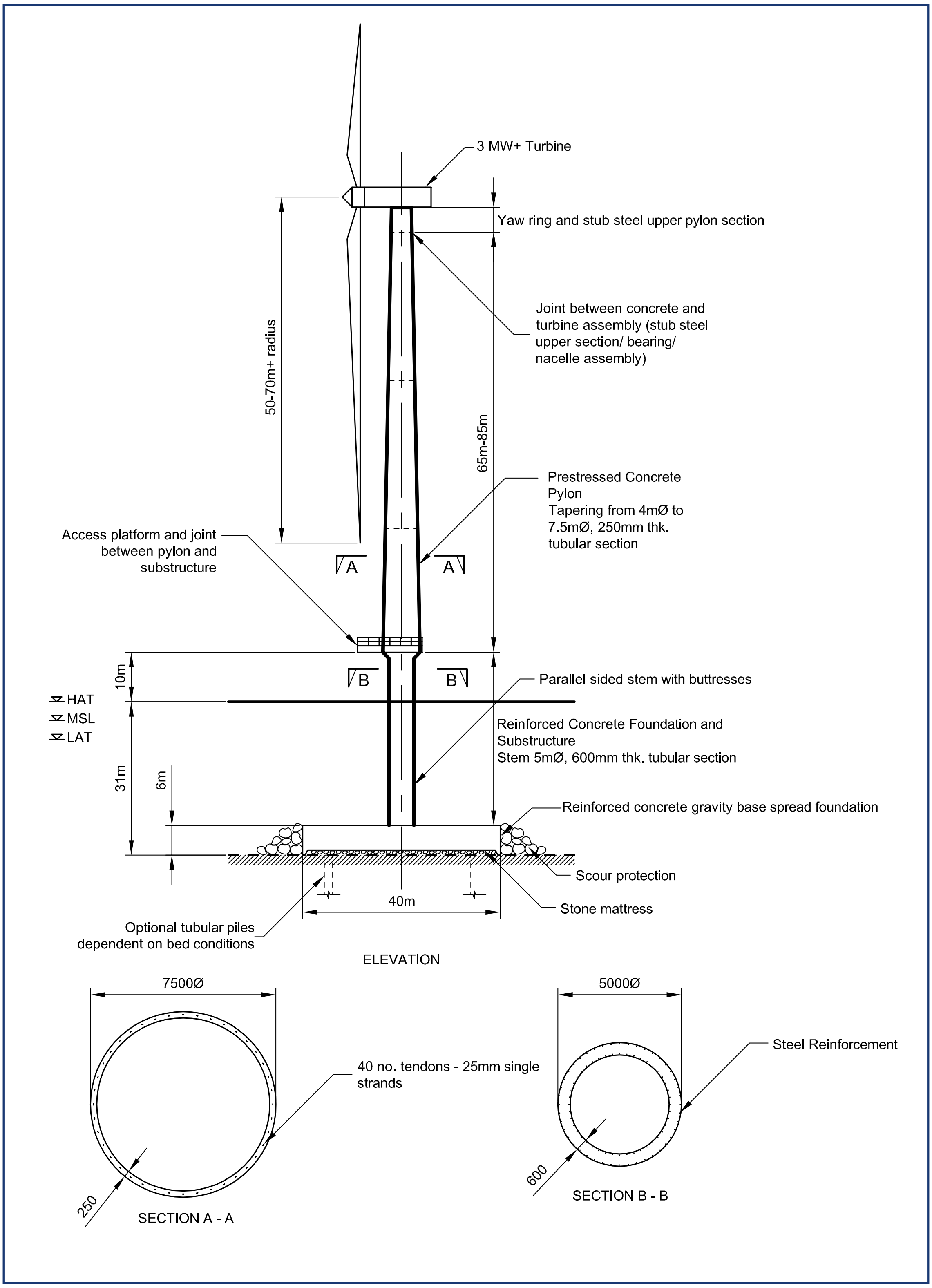


Figure 2.3
CONCRETE TOWER - Necked stem design

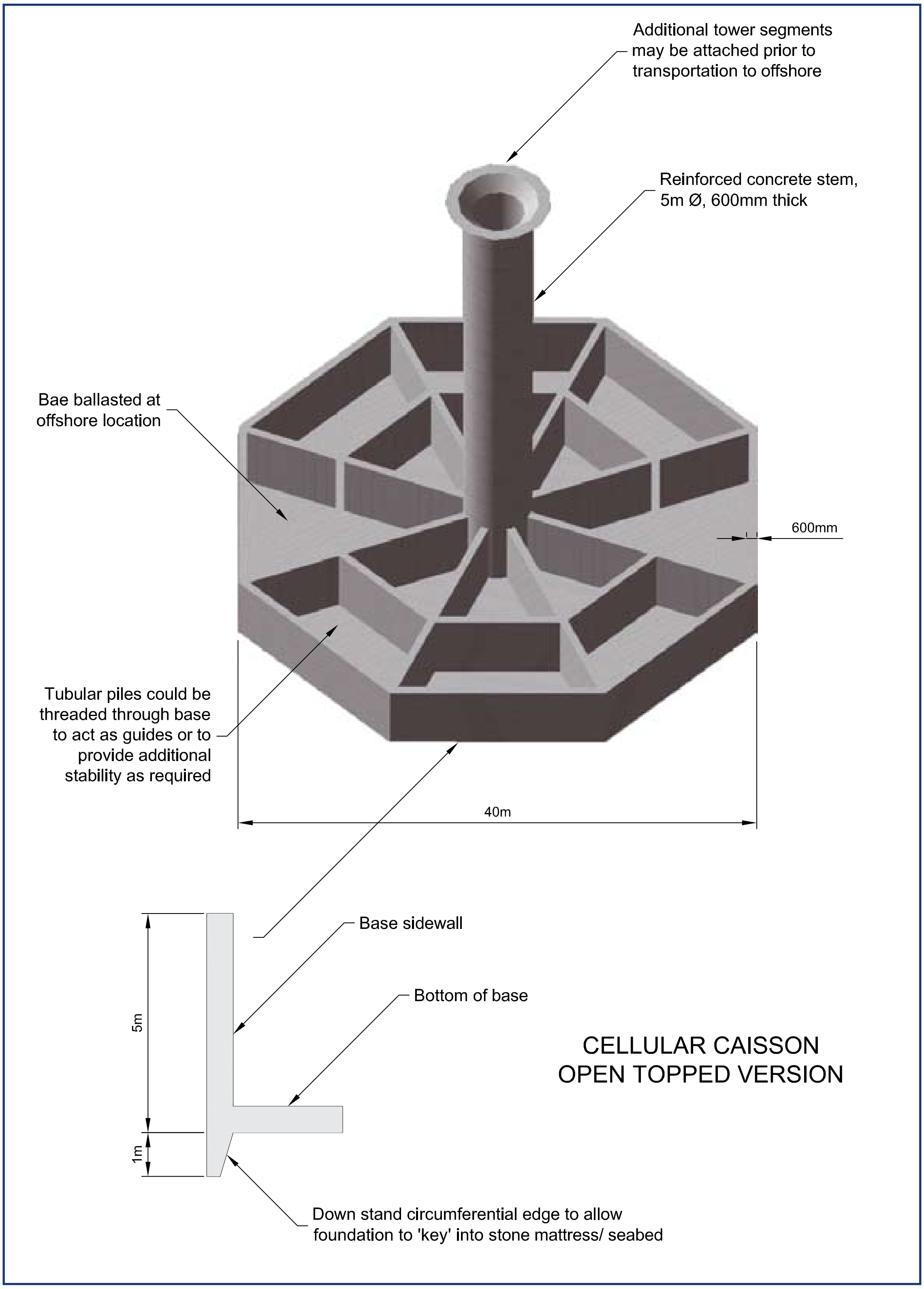


Figure 2.4
CONCRETE FOUNDATION - Gravity base solution

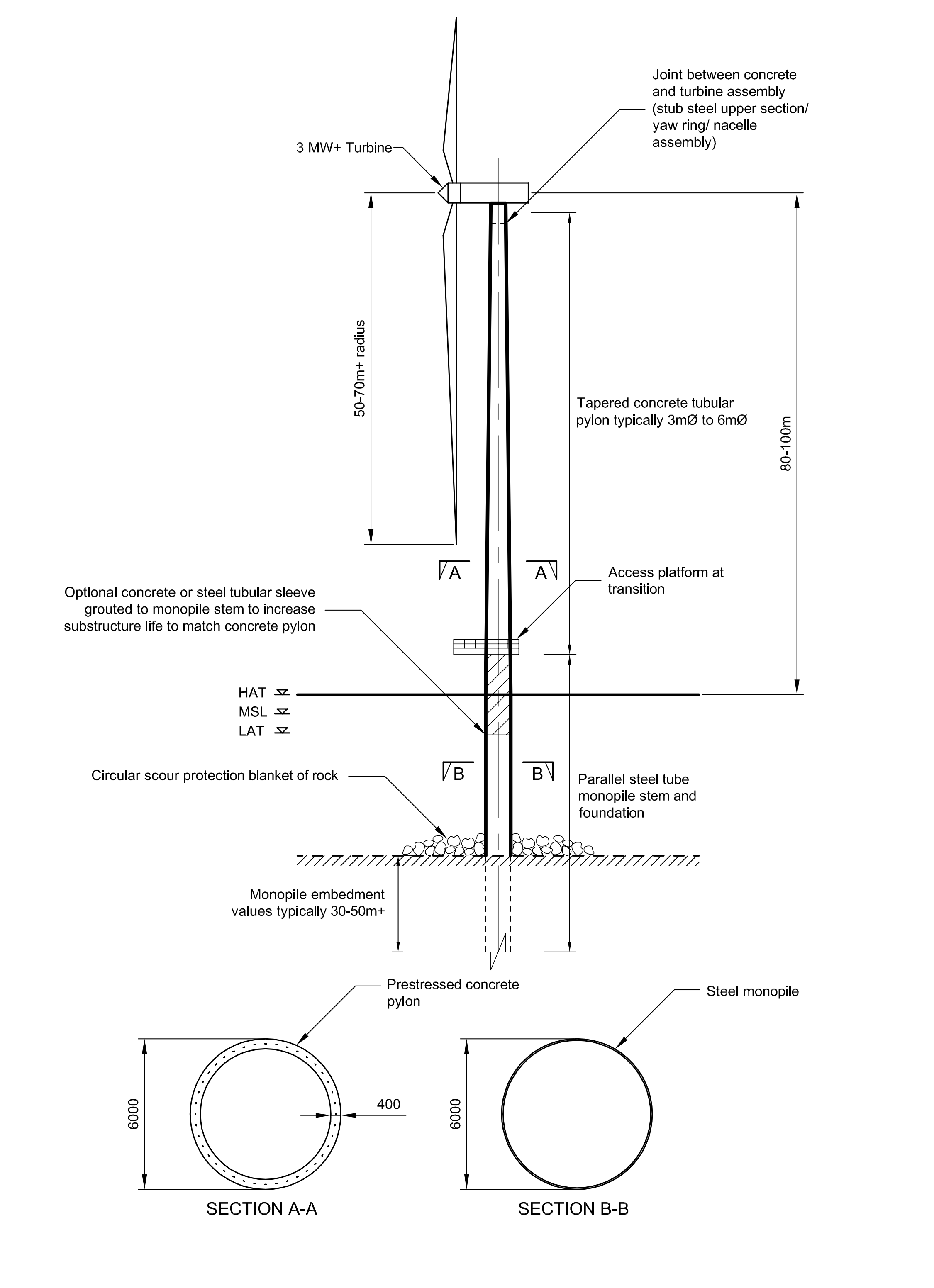


Figure 2.5
CONCRETE PYLON - With steel monopile substructure

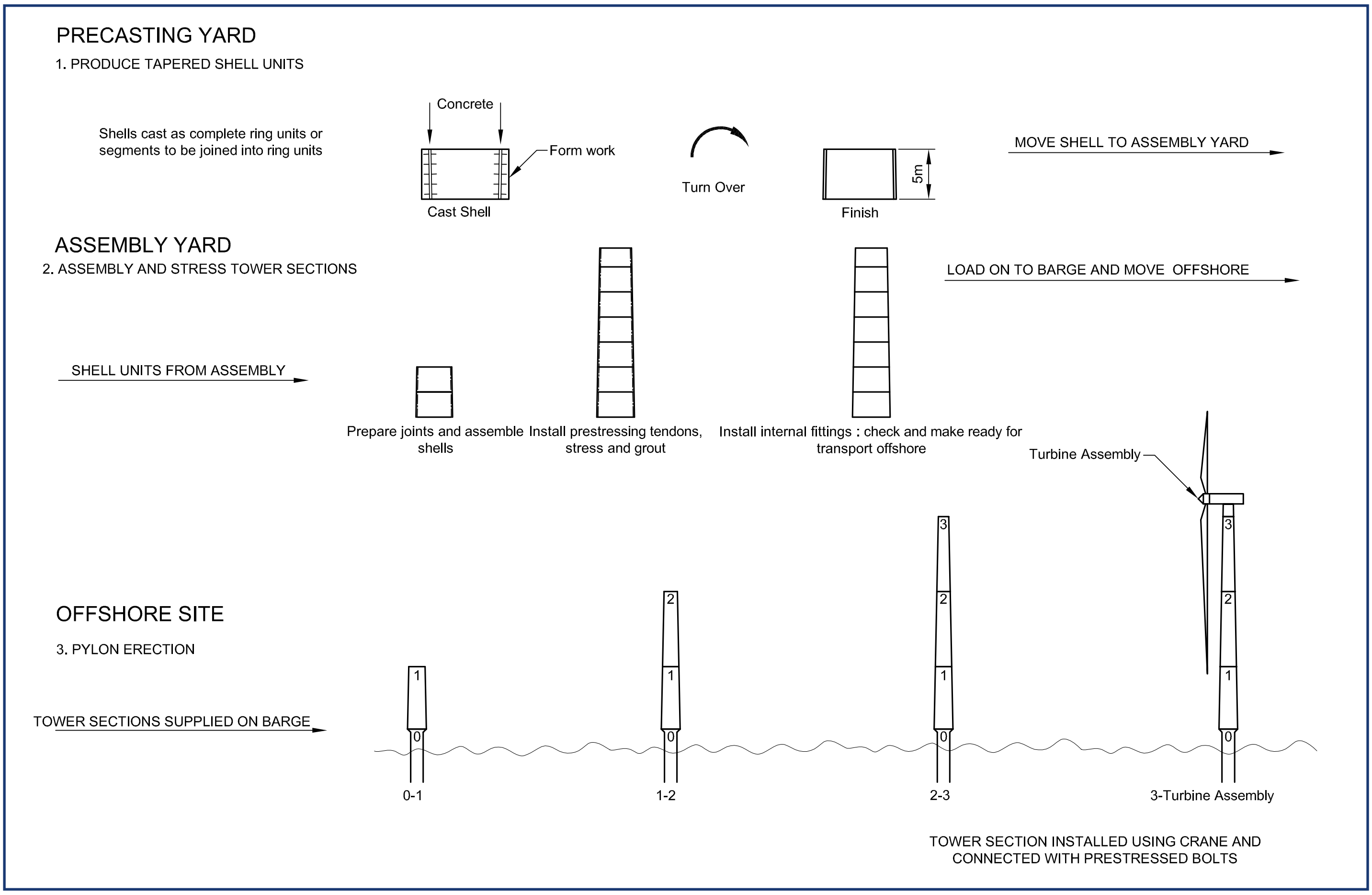


Figure 2.6
PRODUCTION PROCESS - Concrete pylon

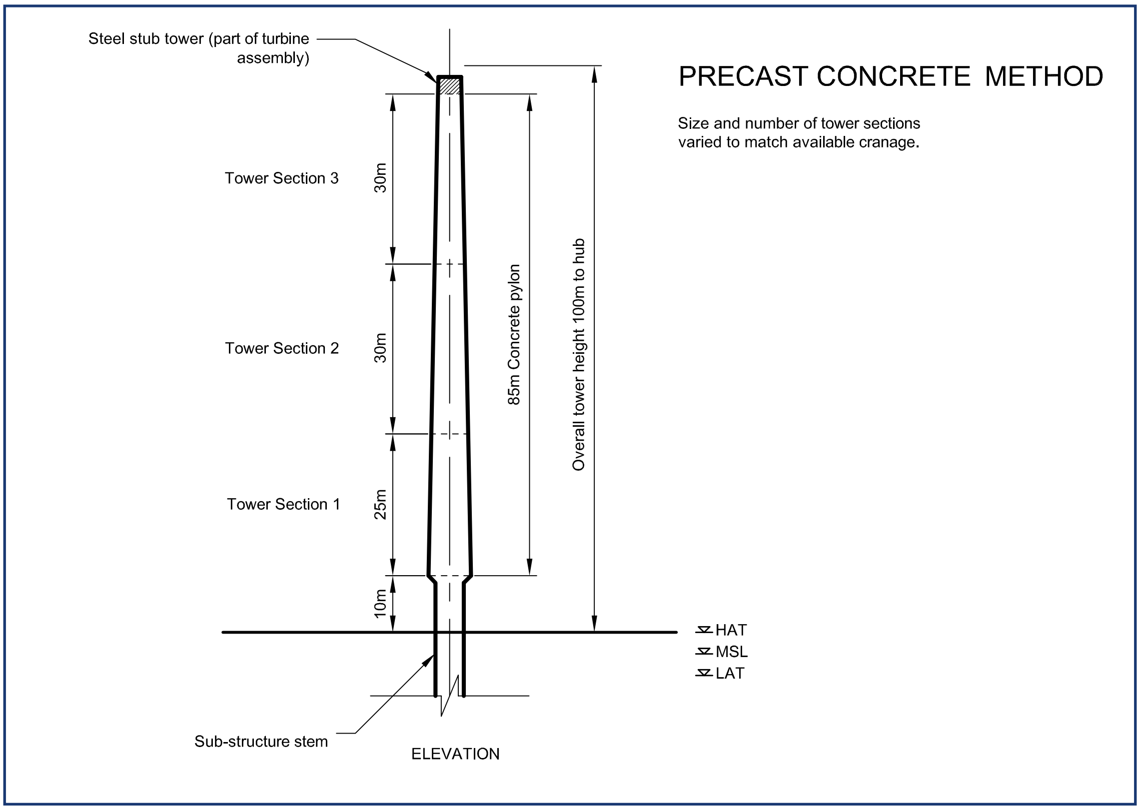


Figure 2.7
TOWER SECTIONS - For offshore erection

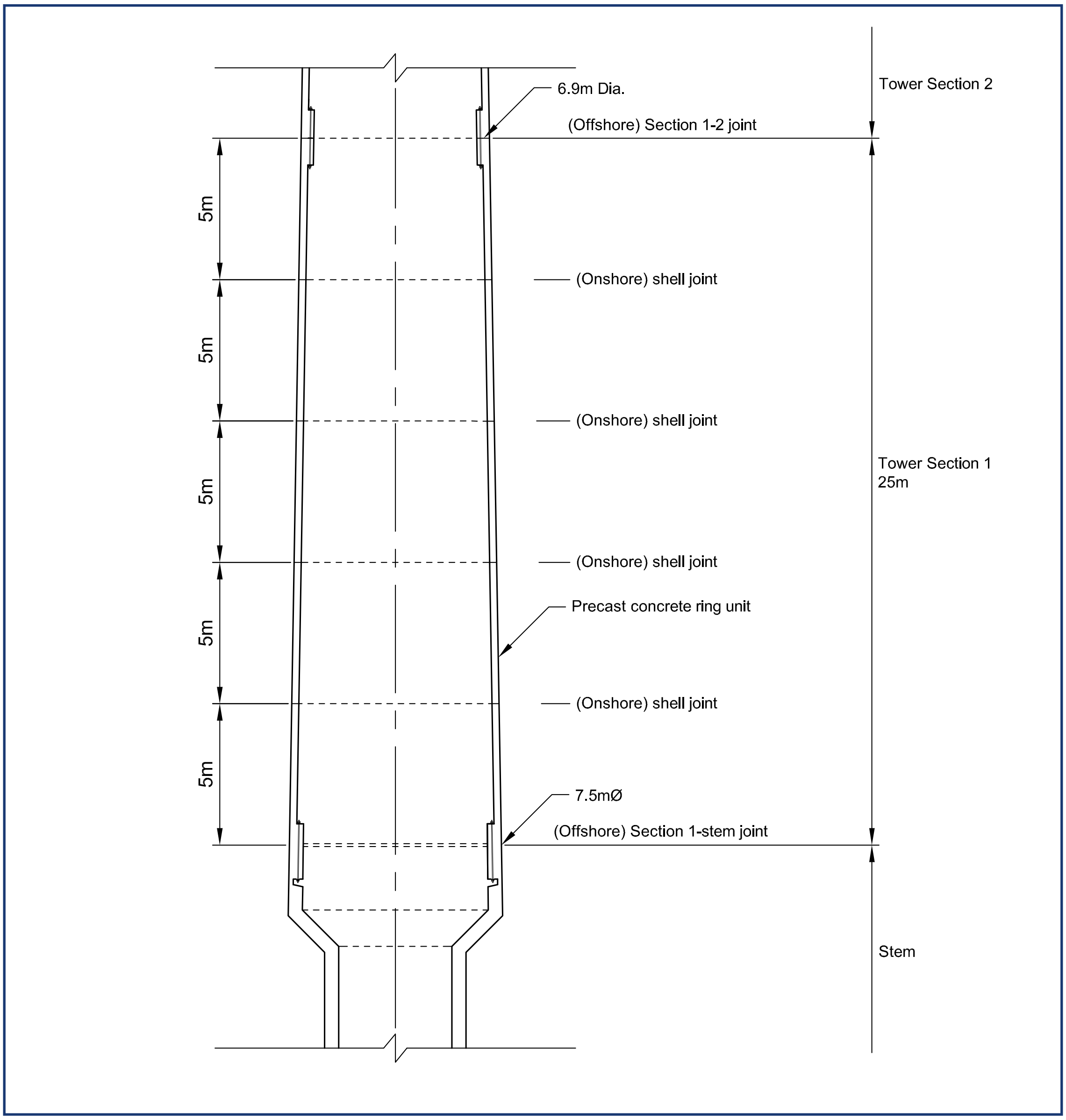
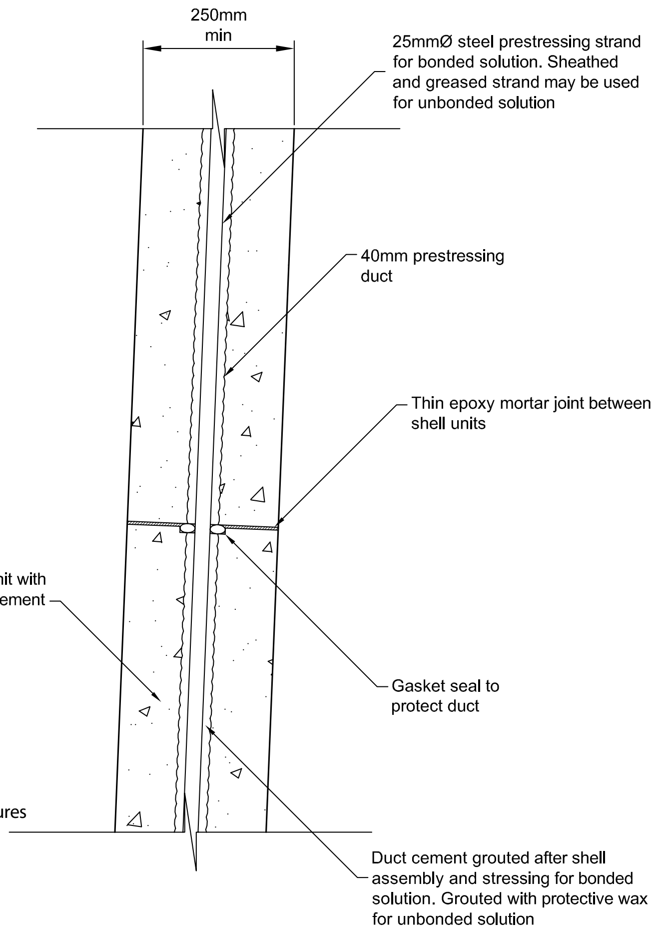


Figure 2.8
TOWER SECTIONS - Showing build-up from precast concrete ring units



For alternative solution using external unbonded prestressing see onshore study figures

Figure 2.9
PRECAST CONCRETE RING UNITS - Outline details

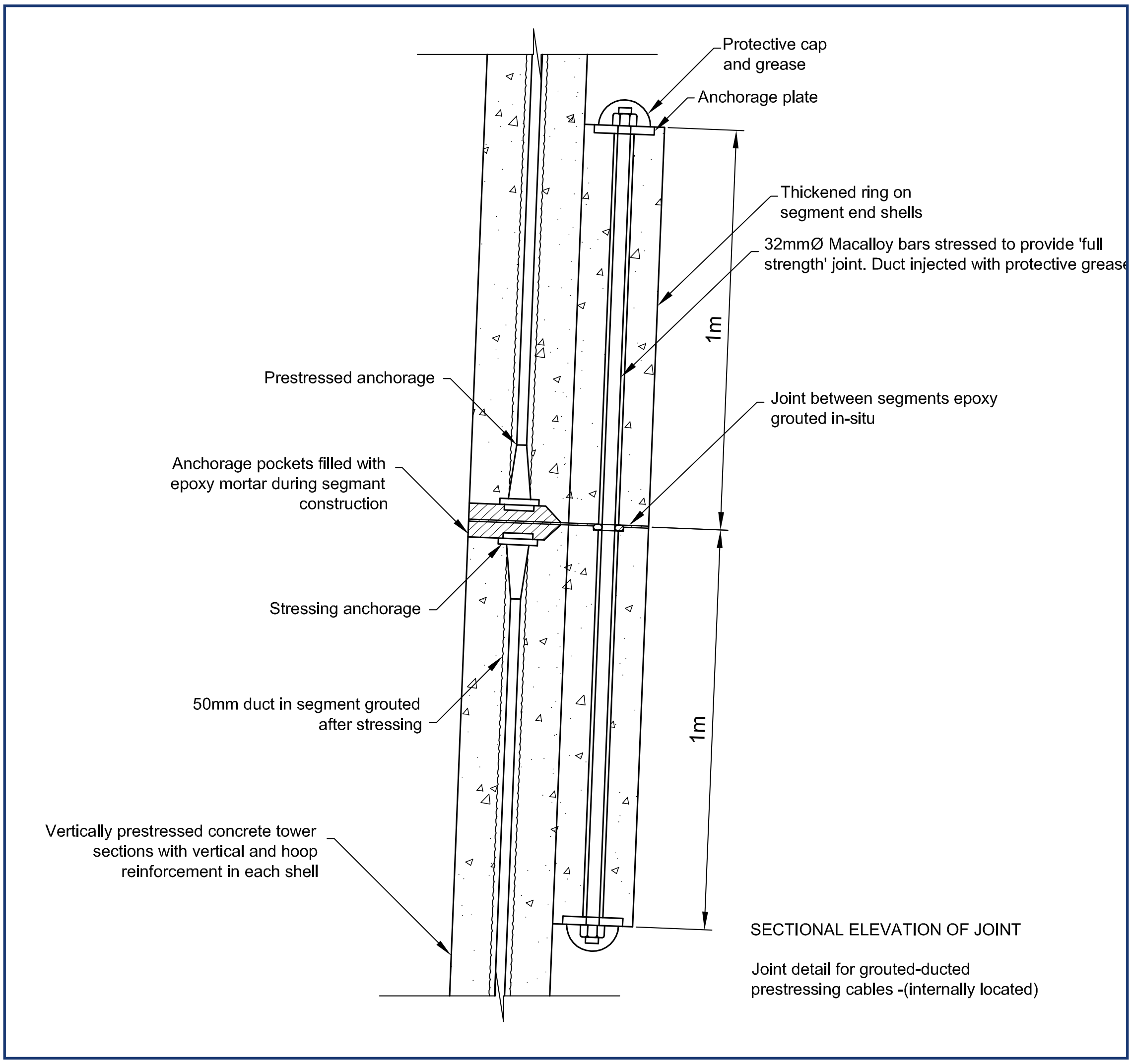


Figure 2.10
JOINT BETWEEN TOWER SECTION - Internal prestress arrangement

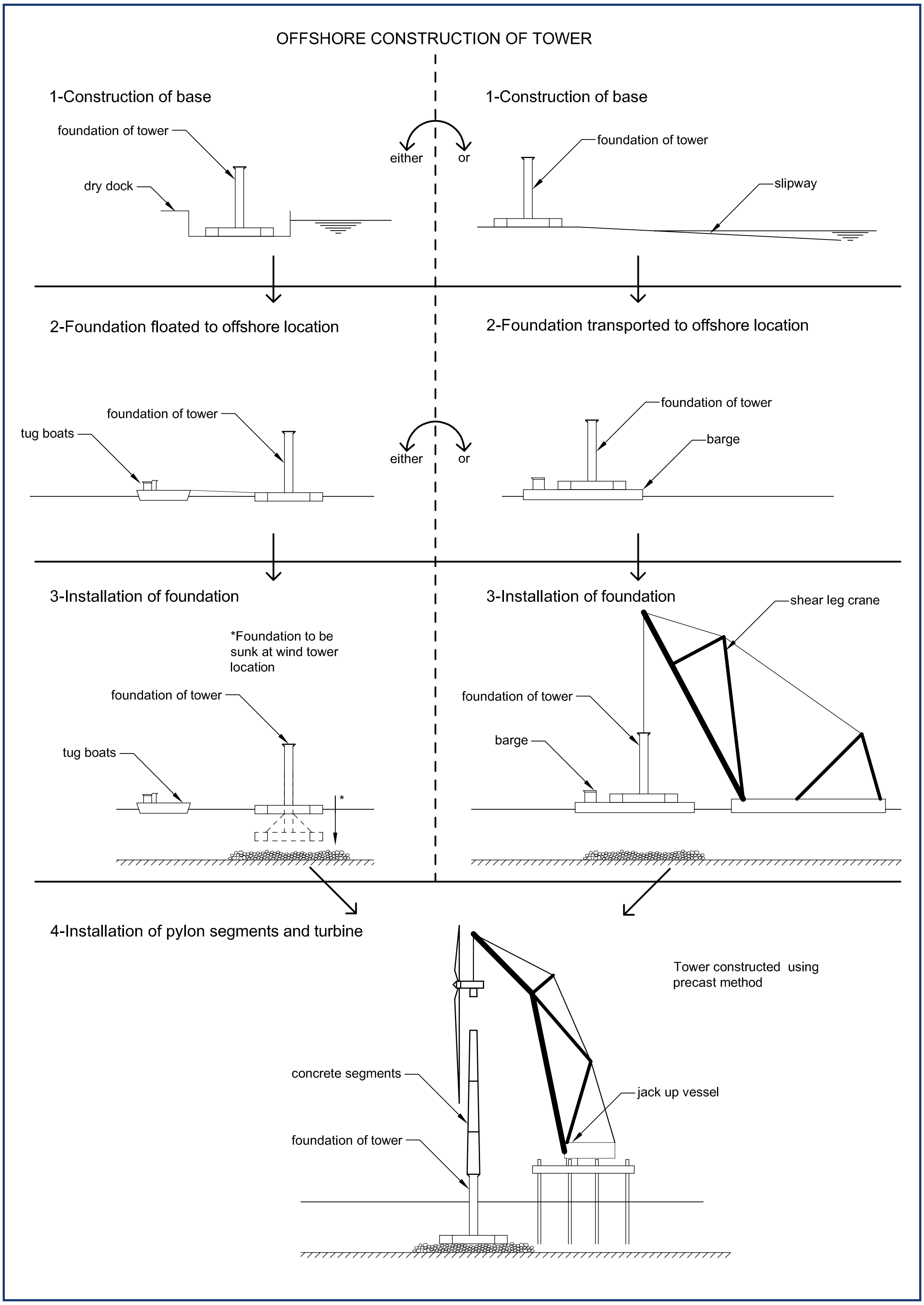


Figure 2.11
INSTALLATION OF CONCRETE TOWER AND GRAVITY FOUNDATION - Methods 1& 2 (Pylon construction offshore)

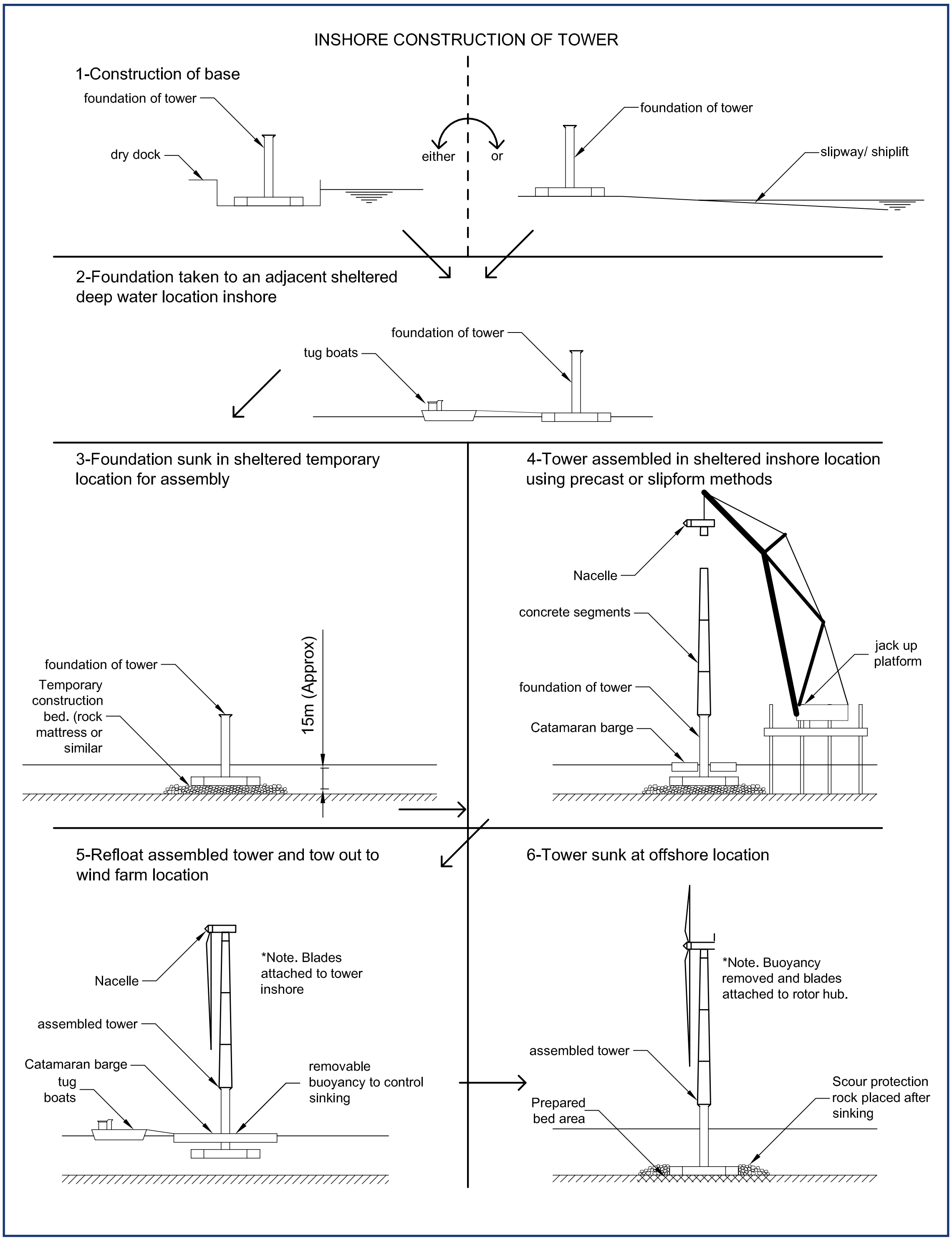


Figure 2.12
INSTALLATION OF CONCRETE TOWER AND GRAVITY FOUNDATION - Method 3 (Pylon construction inshore)

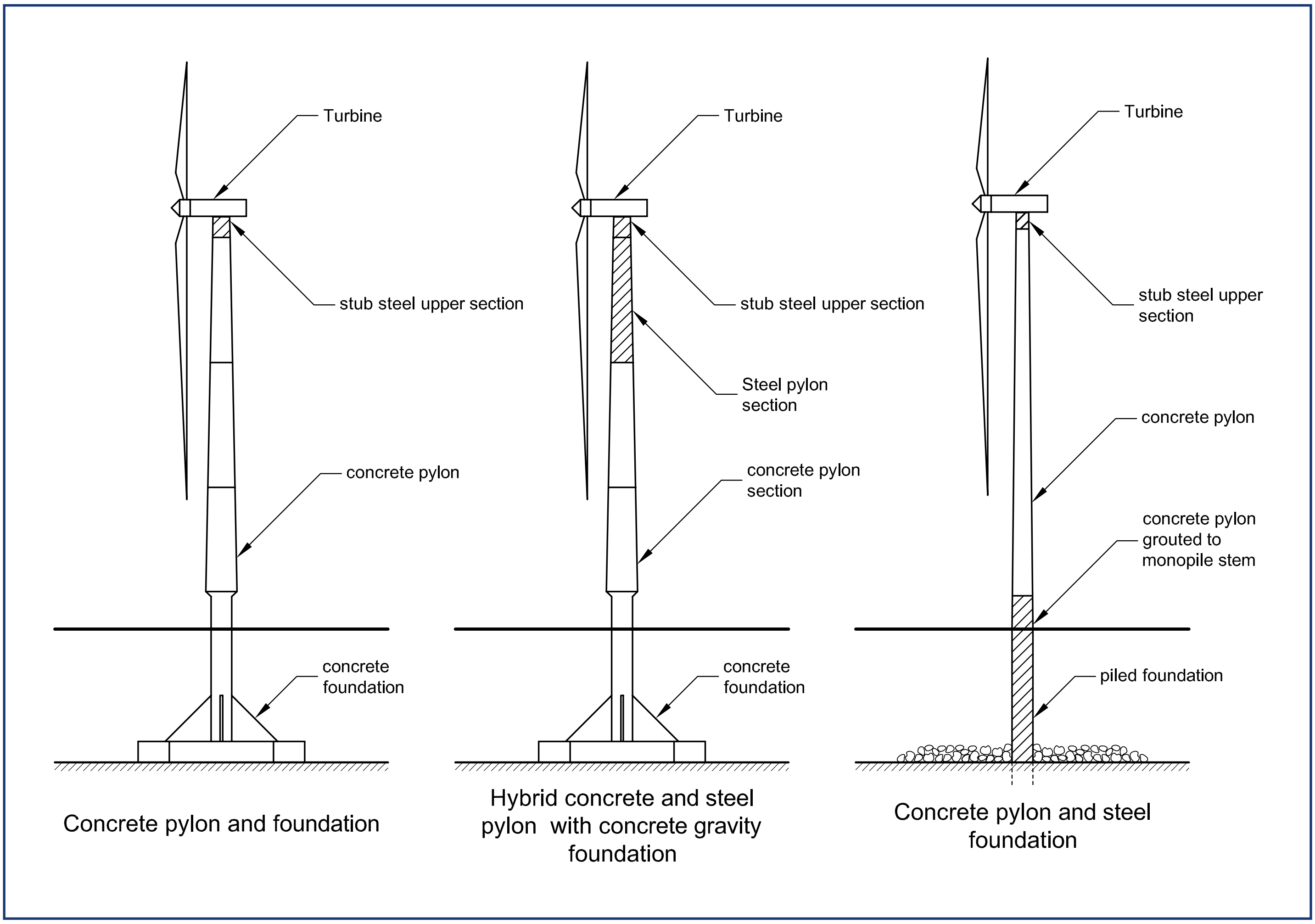
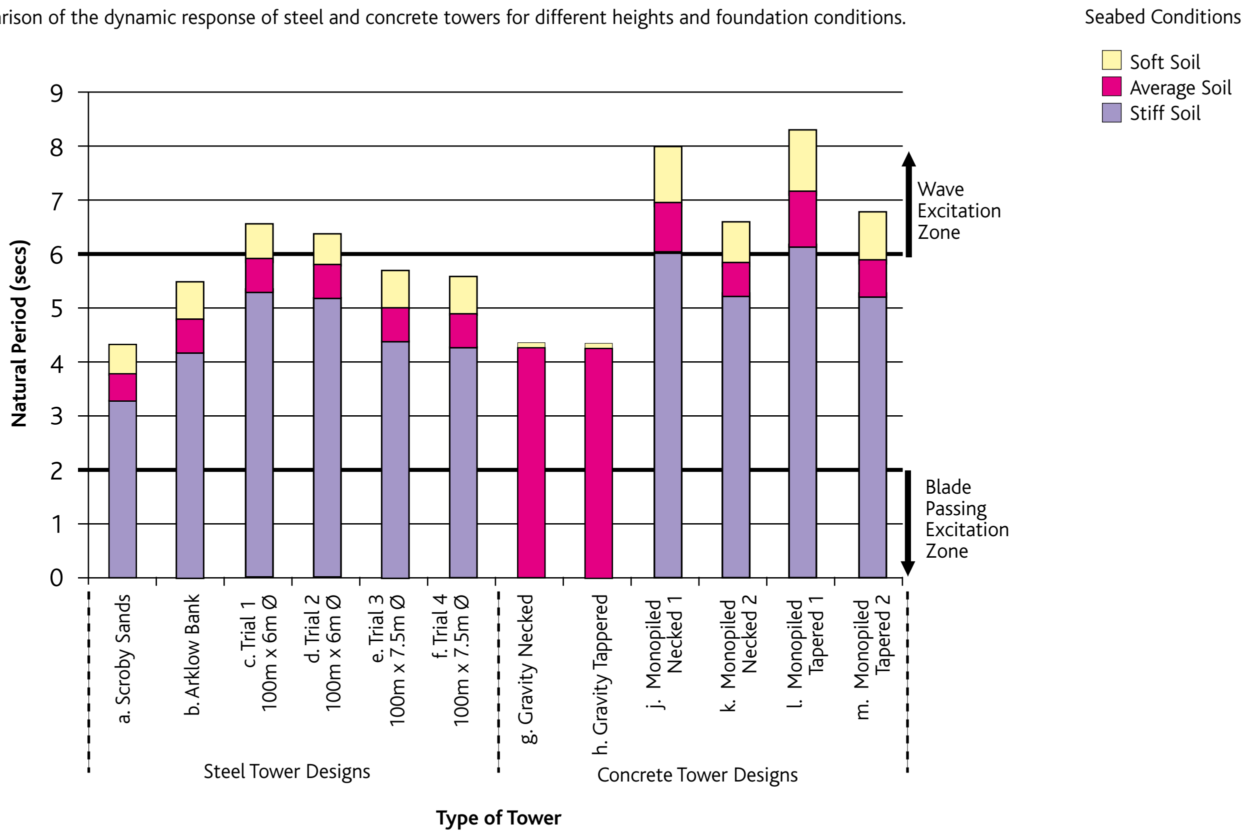


Figure 2.13
COMPOSITE/HYBRID ARRANGEMENTS FOR WIND TOWER STRUCTURE - Flexibility of solution

Comparison of the dynamic response of steel and concrete towers for different heights and foundation conditions.



NOTES ON BAR CHART DIAGRAM

Existing wind farms (Steel monopile solutions)

a) Scroby Sands
Pylon Height – 60m

b) Arklow Bank
Pylon Height – 70m

Theoretical Larger Steel monopile solutions

c) Trial 1
6m diameter steel monopile foundation
3.6MW GE Wind turbine
100m tall pylon

d) Trial 2
6m diameter steel monopile foundation
3MW Vestas turbine
100m tall pylon

e) Trial 3
7.5m diameter steel monopile foundation
3.6MW GE Wind turbine
100m tall pylon

f) Trial 4
7.5m diameter steel monopile foundation
3MW GE Wind turbine
100m tall pylon

Concrete Solutions

g) Gravity Necked (fig 4)
Mass gravity pad foundation
Necked Pylon solution
100 m tall pylon

h) Gravity Tapered (fig.3)
Mass gravity pad foundation
Continual tapered pylon solution
100 m tall pylon

j) Piled Necked 1
5m diameter steel monopile foundation
Necked Pylon solution
100 m tall pylon

k) Piled Necked 2
7.5m diameter steel monopile foundation
Necked Pylon solution
100 m tall pylon

l) Piled Tapered 1
5m diameter steel monopile foundation
Continual tapered pylon solution
100 m tall pylon

m) Piled Tapered 2
7.5m diameter steel monopile foundation
Continual tapered pylon solution
100 m tall pylon

Figure 2.14
DYNAMIC RESPONSES OF VARIOUS TOWER DESIGNS

13 GENERAL CONCLUSIONS

Although some large concrete wind towers have been constructed in Germany and elsewhere in Europe, they represent a small fraction of the present tower population. This report points to the potential for improved competitiveness of prestressed concrete towers, particularly for taller towers and larger rated turbines for both offshore and onshore wind farms.

For onshore wind farms viable designs for both precast and in-situ slipform construction methods are possible.

For offshore wind farms, precast concrete tower designs following current offshore erection methods are considered to be feasible, and possibly competitive. The lower unit construction costs will tend to be offset by possibly slightly longer erection cycles, and the balance of competitiveness could be affected by plant availability and other market circumstances.

Concrete gravity foundations could provide significant benefits and cost savings given appropriate site circumstances, particularly for water depths in the range 15-30 metres.

An innovative approach to construction of an integrated tower/ foundation structure at an inshore site is proposed, followed by transportation and placement offshore of the fully outfitted unit by a purpose designed lift barge. This combines the most economic construction solutions and circumstances with minimum offshore work and risk. This scheme has the potential to reduce the installed cost of the tower structure by as much as 25%.

Further conclusions are:

- Concrete tower solutions are adaptable and durable.
- The high fatigue resistance of prestressed concrete, combined with the good material damping properties of the concrete, can provide a solution which will be potentially less susceptible to and more tolerant of occasional resonance, and therefore have a reduced risk of dynamic problems.
- Given careful design of key details, the high potential durability of concrete can be realised. The tower will need little maintenance during a long design life.
- The use of prestressed concrete can further increase the tower's durability.
- Prestressed concrete towers can be adapted relatively easily to cope with some future increase in loading associated with re-using the tower.
- For taller towers with high top head loading, concrete pylons can be cost-effectively stiffened by enlarging the diameter of the lower tower section above the wave affected zone. This will reduce susceptibility to resonant exposure to excitation by energetic waves.
- Concrete gravity foundations have the potential to give taller towers an improved dynamic response over monopile foundations.
- Concrete solutions are heavier than equivalent steel solutions. Careful selection of section size, concrete strength and the use of prestress can reduce the difference.
- A substantial initial investment in formwork and other equipment will be needed for low unit cost of tower pylons and rapid production. A large enough base market will be needed to fully realise competitive advantages. These investment costs will be larger for offshore designs than for onshore designs.
- The potentially longer service life of concrete towers can be used to significant long term advantage by the adoption of a turbine re-fit programme, involving the replacement of the turbine after the expiry of its (20 year) life. This approach could have a profound impact on the sustainability of the wind energy programme as a whole.

14 REFERENCES

1. BWEA Report, Offshore Wind at the Crossroads, British Wind Energy Association Offshore Wind Conference April 2006

2. Boyle, G, UK Offshore Wind Potential, reFOCUS magazine, Elsevier, July/August 2006

3. Per Vølund, Concrete is the future of offshore foundations, International Conference and Exhibition - Copenhagen Offshore Wind 2005, ENERGI E2, 2005

4. Enercon GmbH, Tower Versions, 2004. Available from: www1.enercon.de

5. Seidel, M, Experiences with two of the world's largest wind turbine towers. Proceedings of the 2003 European Wind Energy Conference and Exhibition, 16-19 June 2003 Madrid, GE Wind Energy GmbH, 2003

6. Brughuis, FJ, Advanced tower solutions for large wind turbines and extreme tower heights. Proceedings of the 2003 European Wind Energy Conference and Exhibition, 16-19 June 2003 Madrid, Mecal Applied Mechanics BV, 2003

7. Brughuis, FJ, 2004. Improved return on investment due to larger wind turbines. Proceedings of the 2004 European Wind Energy Conference and Exhibition, 22-25 November 2004 London. Netherlands: Mecal Applied Mechanics BV

8. See Nordex website: www.nordex-online.com

9. See BWEA website: www.bwea.com

Background reading

BWEA Press Release, Global wind industry grows as positive assessment of grid integration reported from Germany, 2005. Available from: www.bwea.com/media/2005.html

BWEA Report, Onshore Wind: Powering Ahead, March 2006

The Concrete Centre, An estimate of the embodied CO_2 in steel and concrete wind turbine towers, 2005.

Gifford, Review of the estimated embodied CO_2 in steel and concrete wind towers. (Report No. 12318/R02), 2005

Wind Power in the UK, Sustainable Development Commission, 2005. Available from: www.sd-commission.org.uk

APPENDIX A – Embodied CO_2 estimates for wind towers

Table A1 provides comparisons of the typical quantities of CO_2 embodied in structural steel and reinforced concrete. Calculated by independent industry experts, the values reported encompass average CO_2 levels associated with material extraction, production and transportation.

Based on these figures, the estimated total mass of embodied CO_2 in a 70m wind tower constructed from steel and concrete is compared in Table A2. 70m was chosen as a height representative of typical onshore solutions and owing to the widespread availability of industry data on steel towers of this height.

Recognising the indicative nature of these calculations, it is clear that embodied levels of CO_2 in concrete wind towers can be significantly lower than for steel (64% reduction in Table A2).

Concrete offers this improvement as, despite cement production's energy-intensive nature, it is a locally sourced material produced predominantly from materials with low environmental footprints. For further information on the cement and concrete industry's commitment to sustainability, please refer to **www.cementindustry.org.uk** and **www.sustainableconcrete.org.uk**

Additional benefits of concrete to be considered in this regard include:

- The embodied CO_2 values shown in Table A1 are for concrete containing Portland cement CEMI as the exclusive binder material. Using ground granulated blast furnace slag (GGBS) or fly ash as a replacement for cement can reduce the values in Table A1 by as much as 40% depending on the concrete mix design and the application. This CO_2 reduction is in addition to performance benefits inherently associated with these materials including improved concrete strength and durability performance.
- Concrete inherently consumes CO_2 from the atmosphere it is exposed to, a process known as carbonation. Particularly for high strength concrete structures, carbonation levels during service life will be minimal, but if recycled and crushed at the end of its service life, CO_2 uptake is much more rapid. Based on a study by the British Cement Association of all types of concrete, on average concrete can consume 20% of the CO_2 emitted from its cement manufacture [A1].

Table A1

Structural material	Embodied mass of CO2 by mass of material ($kgCO_2$/tonne)			
	Reinforcement content			
	none	low	high	average
Concrete (strength class C32/40) [A2]	140	150	156	153
Structural steel * [A3]	1,932	~	~	~

* Density taken from EN 1991-1-1:2002, Eurocode 1. Actions on structures. General actions. Densities, self-weight, imposed loads for buildings.

Table A2

Wind tower option	Mass of construction materials (tonnes)		Mass of embodied CO_2 (tonnes)
	Structural steel	Reinforced concrete	
70m steel tower *	125	~	242
70m concrete tower **	2.0	550	88

* Estimate using available industry information [A4-A7]

** Estimated based on Gifford conceptual design (average reinforcement level, steel contribution at joints)

Information sources

[A1] C Clear, T De Saulles, Carbonation Scoping Study, British Cement Association, 2007, draft in progress

[A2] The Concrete Centre study, Embodied CO_2 of various concrete mixes, due 2007

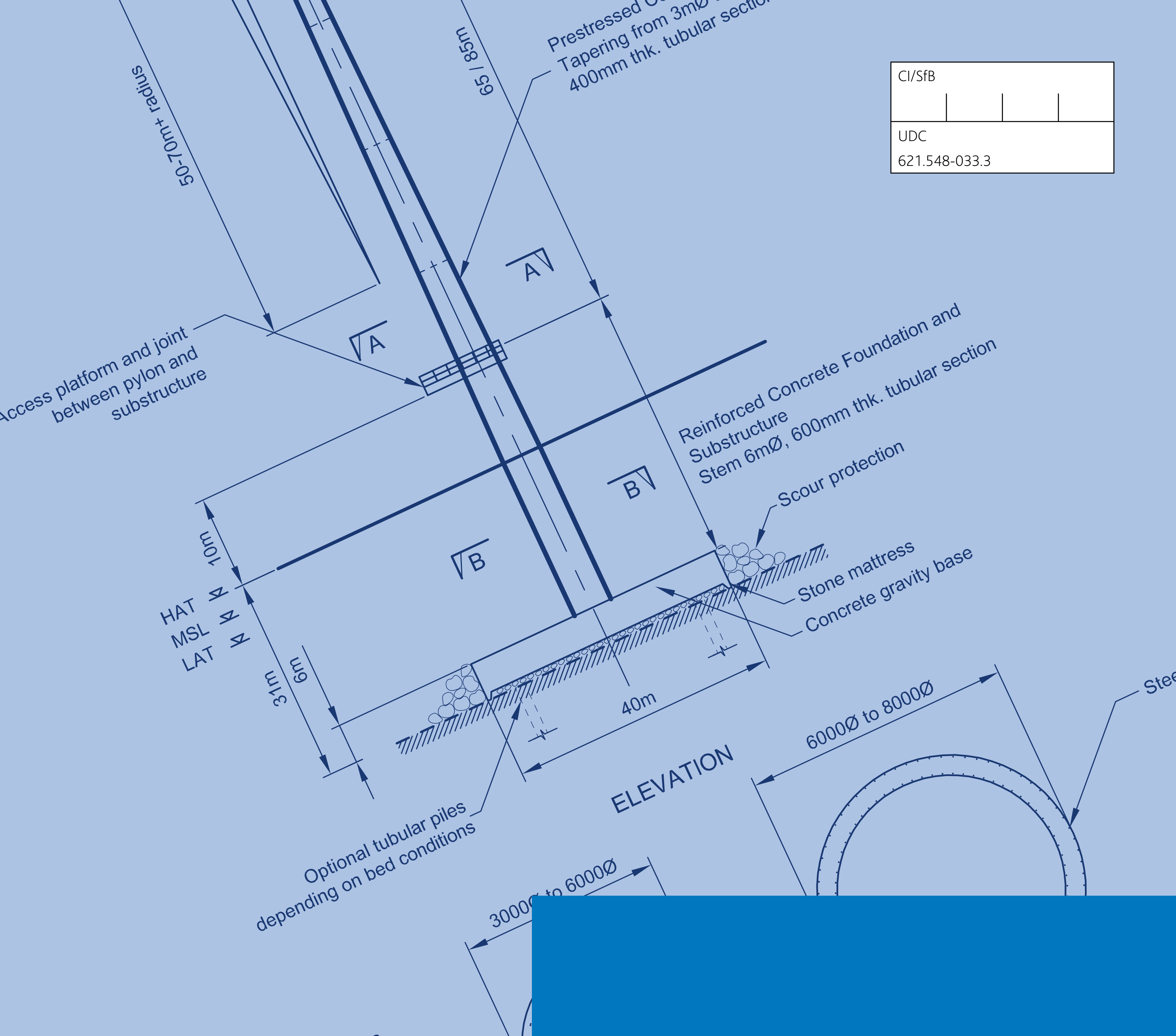
[A3] Amato A and Eaton K J., A Comparative Environmental Life Cycle Assessment of Modern Office Buildings, The Steel Construction Institute, 1998

[A4] Northern power Systems NW 100/19 wind turbine (www.northernpower.com)

[A5] Danish Wind Industry (www.newenergy.org.cn/english/guide/towerm.htm)

[A6] Crokwell, New South Wales 1998 (www.bluecopesteel.com)

[A7] Clashindarroch development (www.amec.com/wind)



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